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2. A 50-word summary must accompany the paper.

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8. Papers should average about 12,000 words in length and must be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

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Proceedings of the American Society of Civil Engineers

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September, 1960

Papers

(over)

DISCUSSION

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

TETRAPODS AND OTHER PRECAST BLOCKS FOR BREAKWATERS

By P. Danel,[1] F. ASCE, E. Chapus,[2] A. M. ASCE, and R. Dhaille[3]

SYNOPSIS

Other shapes have been proposed for precast blocks for breakwaters since tetrapods were first introduced about ten years ago. The authors briefly consider the problems arising in the design, manufacture and placing of such blocks.

Consideration is then given to the problem of utilizing precast blocks on breakwaters. Attention is drawn to the great number of factors that have to be taken into account when designing a breakwater. Serious difficulties may be encountered as a result of neglecting or underestimating the importance of any of the problems referred to.

INTRODUCTION

Over the turn of the half century, some steady progress has been made in the field of coastal structures, which was a branch of engineering, that seemed to be somewhat set in its design outlook. In so far as embankment breakwaters are concerned, it seemed to be normal practice to use cubic or rectangular concrete blocks as a facing, whenever the technical and economic conditions of the site made it impossible to use natural rock. Then, suddenly, about 1950, a new type of facing block was produced which became known as the tetrapod. The design of this block was such that a large reduction in the cost of structures could be forecast, amounting to as much as 30%, in some cases. The fact that tetra-

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pods have been used on an increasing scale ever since would seem to indicate that the claims made for this block were not unfounded.

More recently, other blocks have appeared, among which the tribar in the United States, the stabit in England, and the hollow block (skeleton tetrahedron) in Japan, can be mentioned (Fig. 1). This progress in the field of coastal structures is the result of extensive research and design work using new techniques.

Now (1960) that several years have passed since the introduction of the tetrapod, it may be opportune to review these new techniques, which can lead to such high hopes, and, sometimes, to disappointment. Some of us are amazed that the ideal solution, suitable for all circumstances, has not yet been found, whilst

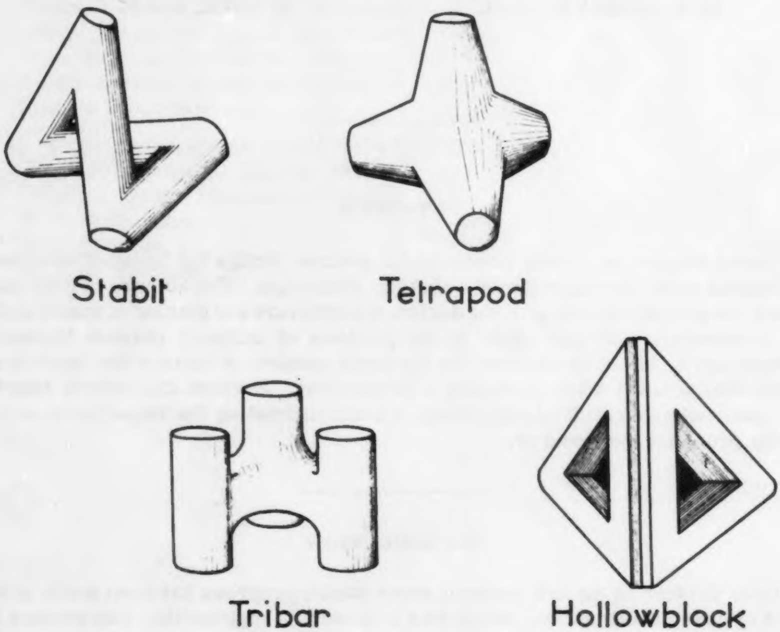

FIG. 1.—VARIOUS TYPES OF BLOCKS

others are so over-confident as to apply new designs before they are fully developed, and still others reject all such new ideas on principle.

The questions involved are extremely important when dealing with coastal structures, as the price of such structures is still very high and any error in the early stages can lead to the most serious consequences. From the financial point of view, it may become necessary to rebuild or repair the structure, while from the harbor traffic point of view protection provided for the harbor installations and shipping may prove to be insufficient. In addition, error in design may lead to the loss of human life.

The problems involved with new shapes of block can be divided into (a) those concerned with the actual block design, and (b) those concerned with the method

of using it on a structure. Some of the essential concepts involved in these considerations will be reviewed to show the difficulties that may be encountered and which have to be solved before undertaking the building of a structure.

By treating the special problem of block design within the general problem of the design and building of the structure, it is hoped to draw some conclusions on the possibilities and opportunities of applying these relatively new products. This appraisal will be based on concrete examples.

PROBLEMS OF BLOCK DESIGN

One reason, and possibly the main one, for the recent progress in the field of coastal structures lies in the great forward strides that have been made over the last few years in hydraulic investigation methods, and especially in the use of scale models.

It is always very difficult to try and observe the behavior of an actual structure during a storm. It is almost impossible to try and measure block movements and record pressure variations, both near and within the structure under these circumstances, and one usually has to be content with inspecting the state of the structure before and after the storm. It is usually out of the question to adequately measure, or estimate, the characteristics of the storm itself. However, our knowledge of the sea's behavior has greatly improved, especially as regards methods of forecasting waves from meteorological data. These methods are an extension of those developed during World War II, in connection with preparations for various landings.

At the same time, a great deal of work has been done in the analysis of wave phenomena and the changes that take place near the shore and around coastal structures, as well as in the analysis of the forces acting on the facing blocks. The invention of the test flume wave filter, by Mr. F. Biesel, in 1947, made it possible to investigate hundreds of structures in the laboratory with standards of phenomena reproduction, and with a saving of time and money, never before achieved.

With this improved knowledge of natural phenomena and with more modern methods of research, test establishments have been able to analyze the behavior of orthodox structures under wave attack, and discover the deficiencies of the then current types of block. It was then logical to find out what the basic requirements of an ideal facing were and consequently develop new block designs. The result has been the appearance of blocks of novel design that have found immediate applications. Further development is leading towards still more revolutionary designs.

Therefore, it is due to the hydraulic laboratory that such rapid progress has been possible in the field of coastal structures, after so long a period of slow developments. This is especially true as regards the design of the new blocks that have appeared. Indeed, it would sometimes seem that the basic part played by the specialized hydraulics engineer has led to a tendency to only consider the hydraulic aspect of the problem, and to be satisfied with a block that gives satisfactory results when subjected to model waves alone.

Although the hydraulic aspect is obviously of the highest importance, there are other questions involved, including the full-size manufacture of the block, placing it on a real life structure and making it perform satisfactorily over a considerable number of years. Such practical questions as manufacture, handling and placing, mechanical strength and corrosion resistance are problems which,

if ignored, would soon justify the opinion of those who consider these innovations to be nothing more than laboratory toys.

The tetrapod block is given as an example of various problems encountered during development of the various blocks and how these problems were overcome. The same problems have to be solved with any other block.

Hydraulic Problems.—The authors will not dwell on the hydraulic considerations which led to the tetrapod shape. Suffice it to say that once it was realized that a shape having a central hub with a number of projections would give considerable improvement over all other known shapes, it was then possible to experiment with the number and shape of the projections. The hydraulic tests showed that there was an advantage in lengthening the projections in order to improve both the interlocking capacity of the blocks with each other, as well as the energy dissipating efficiency of the facing (Fig. 2).

However, at this stage of development, hydraulic considerations ceased to be the all-important questions and were superseded by the other factors previously mentioned. In fact, it is no exaggeration to say that the final shape of

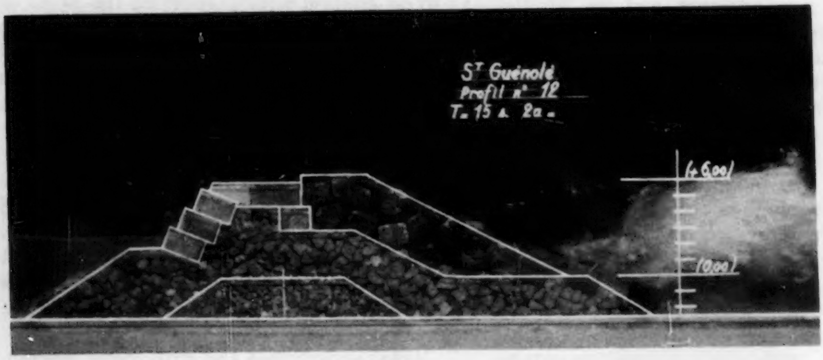

FIG. 2.—SCALE MODEL TEST OF ORTHODOX DESIGN

the tetrapod was arrived at through a strength of materials consideration rather than a hydraulics consideration.

Mechanical Strength.—The important problem in mechanical strength is to design a block with a resistance to breaking, both during manufacturing and placing and also during any possible movement after placing, such that it will not sustain any damage likely to alter its hydraulic characteristics. As far as the tetrapod block is concerned, the design studies were backed up with breaking tests in both the laboratory and also to full scale.

The similitude laws for rupture are simple. If a is the length ratio and b is the fall height ratio for two blocks with the same shape then; the fall energy ratio is a^3 b; the elastic displacement ratio at the time of rupture is equal to a; the force ratio is a^2 b; the moment ratio is a^3 b; the opposing moment ratio is a^3; and then, the stress ratio is b. This means that, if blocks of different sizes are made of the same material, failure will occur for the same fall height irrespective of the block size. This law was confirmed by tests carried out on 295 tetrapods ranging in weight from less than 20 oz to 25 tons.

These tests were carried out using various concrete mixes, with and without reinforcing, by drop tests on to a rock bed, a highly reinforced concrete slab, and, finally, on to a concrete slab with a varying depth of water cushion (Figs. 3, 4, and 5).

The results, which showed good agreement, can be summarized as follows:

1. With a rock bed, no damage was sustained by the tetrapods when dropped from heights of up to 10 ft, which was the maximum height the test rig would allow.

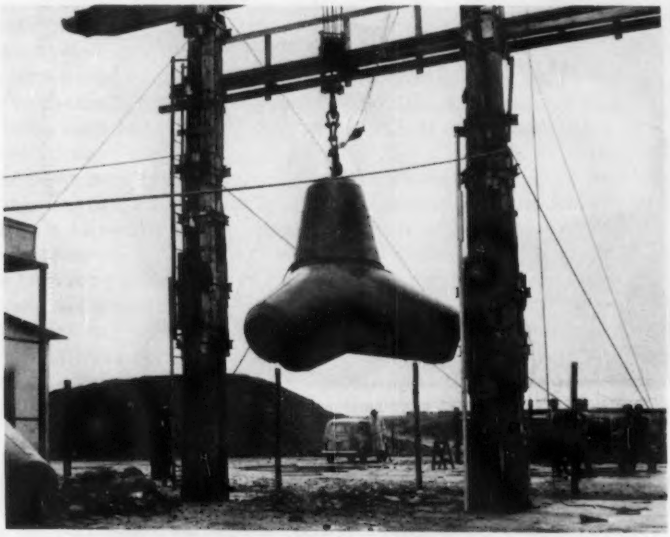

FIG. 3.—DROPPING A 15 TON TETRAPOD

2. With a non-reinforced tetrapod dropped directly on to a highly reinforced concrete slab, fractures were observed with a drop height of a little over 4 ft.

3. Tetrapods reinforced at 55 lbs per cu m, when dropped on to a highly reinforced concrete slab, showed deep cracks with a drop height of a little over 4 ft.

4. A water cushion of about 1/6 or 1/5 of the tetrapod height prevented any damage to the tetrapods when they were dropped from 4 ft on to a reinforced slab.

The practical results of these tests are as follows:

1. During handling on the site and laying above or, more especially, below the water level, it is most improbable that a nonreinforced tetrapod will break except in the event of an accident. In fact, if the impact velocity associated with a fall from 4 ft, which is the most difficult case as it involves a shock against a practically irresistible mass, is compared with the normal rate of lowering a block by crane, it will be seen that there is a very large safety margin.

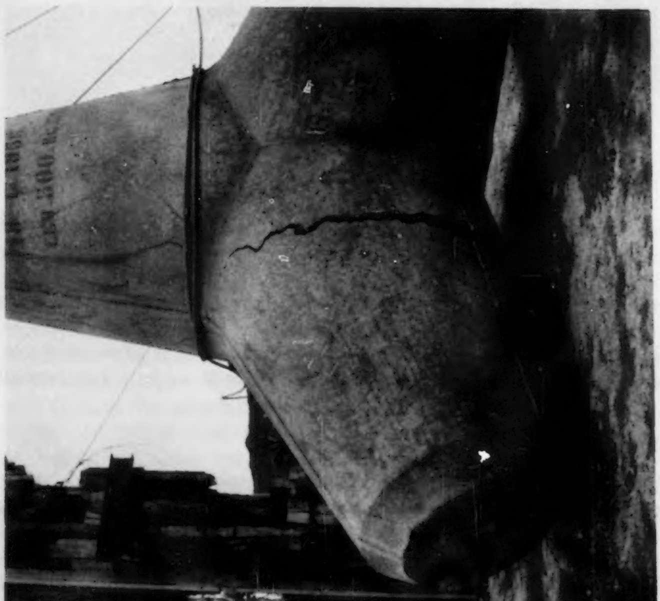

FIG. 4.—RESULT OF 4 FT. FALL ON CONCRETE SLAB

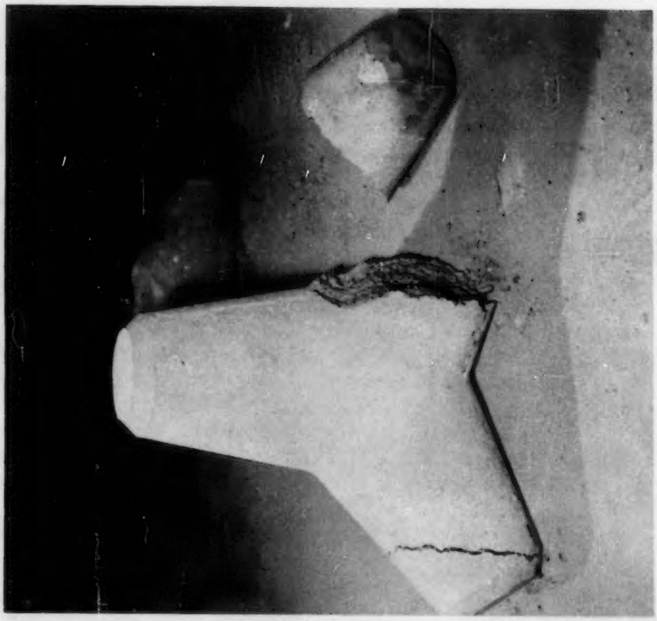

FIG. 5.—BREAK OF A 500 LB. TETRAPOD

2. Should the blocks move when the first storm occurs, before they have finally locked themselves together, there is no chance of breakage as such movements occur under water, and are, in any case, of small amplitude.

3. The fitting of relatively light reinforcing is of no great advantage since any cracks that appear would certainly result in the reinforcing being attacked by the sea water, so that the tetrapod would not only eventually break into two pieces, but corrosion would continue to attack the reinforcing, and, eventually, affect the whole block.

These conclusions have been confirmed by site experience and, with some 55,000 tetrapods actually placed in many sites in the world, breakages have been at a rate of 1 or 2 per thousand.

Manufacture and placing.—At first sight, the tetrapod shape appears complicated and would seem to offer some interesting molding problems. However, there are actually only several possible solutions. A number of molding methods have been used but only two are now currently used and these are briefly described here.

The first involves the use of a two-piece metal mold. This method simplifies handling of the molds since it is only necessary to disconnect the two halves, which are joined by a few clamps, and lift off the top portion. Practically speaking, however, there are a number of difficulties involved in doing this because it is very difficult to lift the mold exactly along the center line of the upright leg, and also to pull it off progressively enough. The result is that the concrete, which has only been set for 24 hr, is subjected to transverse and tensile loads. Small cracks were frequently observed, and sometimes, pieces, actually broke off. This was remedied by using small screw-jacks to start the mold lifting operation. However, it seems that this additional operation tends to detract from the advantages of ease of handling associated with the two-piece mold (Fig. 6)..

The second and most common method in use involves the use of a mold built up from four identical sections, joined by lever or screw clamps. The complete mold can be considered as a hollow body with an inherent amount of elasticity, such that it actually expands slightly when it is filled with concrete. Although additional work is involved in taking off the greater number of clamps, there is absolutely no risk involved in removing the mold, and the finished surface is absolutely perfect (Fig. 7).

After drying for a period of from 48 hr to 72 hr, according to the size, the tetrapod is lifted out of the bottom mold by means of a special sling consisting of three plates, that rest against the ends of the base legs, and are joined by a cable. In this way, the whole concrete form is in compression, so that there is absolutely no risk of cracks forming in the relatively fresh concrete and, at the same time, the lifting eye becomes redundant.

No steel at all is used for reinforcement or for lifting eyes. Corrosion is no longer a problem and costs are reduced to a minimum (Fig. 8).

Tetrapods are placed by means of a sling and a simple auxiliary cable arrangement, that serve the dual purpose of guiding the tetrapod into place and releasing the sling once the block is resting on the structure. The arrangement is such that it cannot open under load, thus affording an additional safety factor against accidental dropping (Fig. 9).

As an indication of the ease with which tetrapods can be handled, site experience has shown that lifting a tetrapod from the bottom mold takes about 1 min. and the rate at which the blocks can be placed on the structure is about one

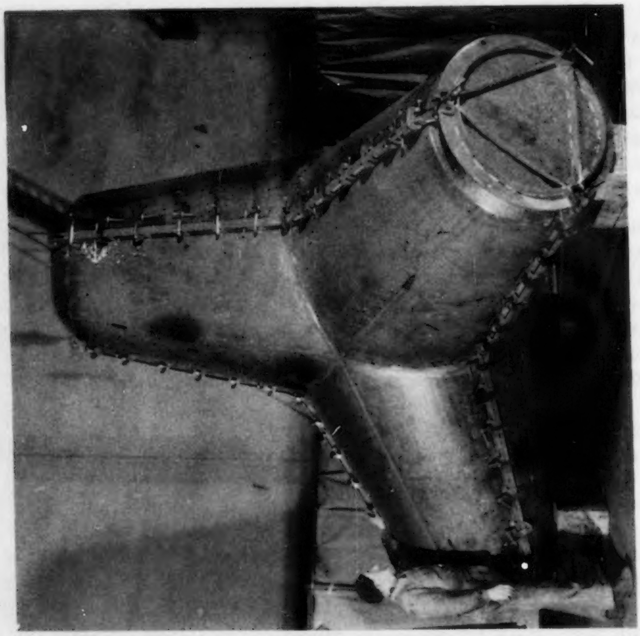

FIG. 7.—FOUR PIECE STEEL MOLD

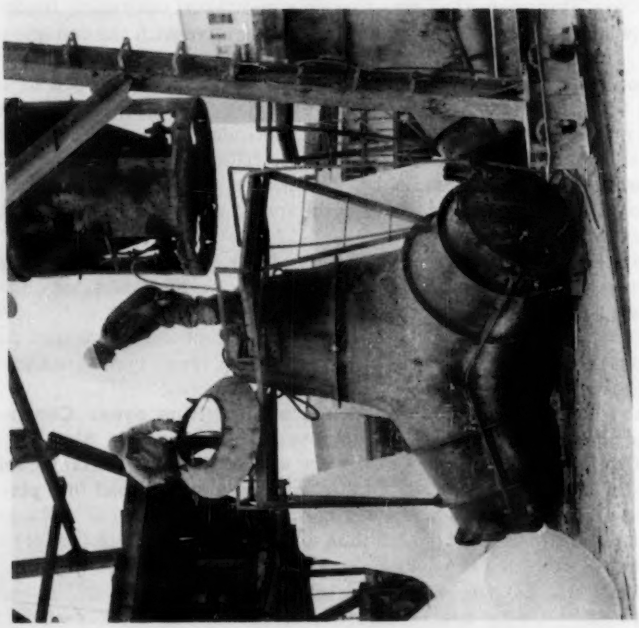

FIG. 6.—TWO PIECE STEEL MOLD

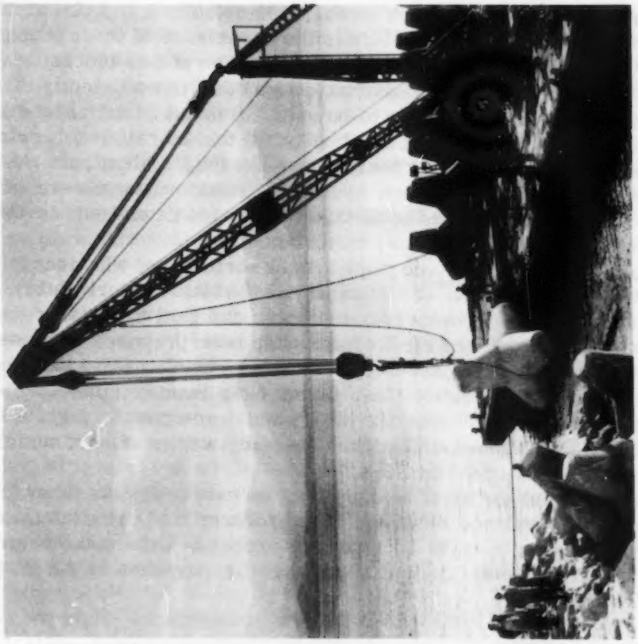

FIG. 9.—PLACING A TETRAPOD

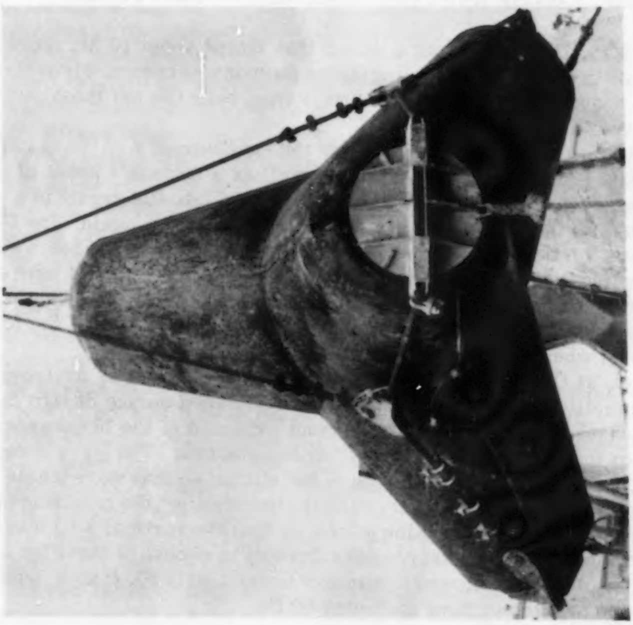

FIG. 8.—LIFTING DEVICE

every 3 mins. These questions of block strength, manufacture, and placing have been treated in some detail in order to stress the importance of these problems when it comes to considering the practicable application of new blocks.

The most essential factors are to have a block (a) that is sufficiently strong to meet the conditions under which it is to be used, (b) that will not corrode, (c) that is easy to manufacture and place, and finally, (d) that is reasonably priced, in all of which the very best hydraulic design plays not the slightest part. As far as the tetrapod is concerned, although hydraulic considerations were of the highest importance, and were treated as such, the other problems could certainly not be considered as being of secondary importance.

All this appears quite obvious if one stops to think seriously of what is involved in the construction and durability of a coastal breakwater. Unfortunately, the general enthusiasm resulting from a new discovery can lead to certain factors being overlooked. Newly invented blocks have often been proposed for general application, in spite of the fact that only very few hydraulic tests have been carried out and the other aspects of the problem have been completely overlooked. Like so many other problems, the design and development of a new type of block is basically a question of finding the best compromise, after a complete evaluation of all the various possibilities.

It is not surprising that the ideal design is not as easy to find as many of us would like to think. Even when a block has been produced that satisfies the hydraulic requirements, the strength of materials aspect and the manufacturing needs, the block still has to be applied to the specific purposes of the project authority concerned.

PROBLEMS OF APPLYING BLOCKS TO STRUCTURES

On February 19, 1955, following a storm that lasted about 10 hr, more than 2000 ft of the main breakwater protecting Genoa Harbor was completely destroyed The breakwater concerned was of the vertical type, built for the most part with 400 ton concrete blocks.

The immediate result of this disaster was the explosion of a cargo of carbide on board a vessel anchored in the harbor, as well as a certain amount of other damage that the Genoa Harbor Authority was able to isolate and repair in a most effective manner, so that harbor traffic was soon able to flow again. The Genoa breakwater was a vertical jetty whose design fully complied with modern standards, and according to the waves for which it was designed, it should have stood up to this storm without damage. When dealing with a vertical breakwater, full design computations are possible including an accurate analysis of the forces acting on the wall when it is subjected to a pure wave.

In actual fact, at Genoa, it is probable that the structure was destroyed by waves with an amplitude only a little over that considered during design calculations. Unfortunately, the total reflexion effect expected of the breakwater did not occur, due to the following exceptional circumstances: The wave direction changed between the beginning and the end of the storm, so that wave trains with different periods and directions converged on the breakwater; the resulting interference produced a system of breaking waves, so that the vertical wall was subjected to direct impacts with pressures considerably in excess of those for which it was designed; photographs showed columns of water nearly 300 ft high, whereas the total reflexion should not have exceeded 60 ft.

This example is given to stress how much more difficult it is when an embankment structure is involved, where the mathematical analysis of the forces involved is at the most, very incomplete. Here again, scale models can be a great help to the engineer by giving a general result where analysis is impossible.

By this means, the stability conditions for a facing of given characteristics can be determined and, be general tests on facing of simple form, characteristic curves for the facing, when built with the particular blocks in question, can be compiled. However, it is insufficient to consider the problem solved, simply by knowing the four parameters corresponding to a point on the stability curves. These parameters are wave amplitude and period, unit weight of the block, and the slope of the facing.

Actually, systematic tests on facing stability are generally carried out under simple conditions with any peculiarities avoided as far as possible, that is with the structure sited in very deep water, the facing extending well down and continuing above the maximum height reached by waves breaking on the structure. This procedure is quite vital in order to get results of a more general nature and enable comparison to be made with other facing methods. However, the problem becomes a great deal more complicated when all the other factors involved with an actual structure have to be introduced, including in particular; (a) tide, (b) the possible rise in the mean sea level as a result of local depressions, or water build up at the coast as a result of wind, (c) the wave characteristics, (d) the water depth and the sea bed contours, which will determine the possibility of wave concentrations, (e) the type of sea bed, and the problem of the structure's foundations involved therewith, (f) the meteorological conditions over the whole wave generation area, indicating whether any catching up or concentration effects are likely, as was the case with Genoa, (g) the chief purpose of the structure and the degree of protection required, including such thing as the stillness of the harbor water, the overtopping limit, the limit for slight damage and the destruction limit, (h) the navigation requirements in front of the structure, which can determine the permissible reflexion coefficients, (i) the building conditions on land or in water, according to the number of days when the sea allows work to be carried out, (j) the existence of provisional harbor structures, to protect construction work, and (k) the presence and possibilities of local quarries. This is by no means a complete list, but it is quite sufficient to show that combinations of these various factors can lead to an almost infinite number of problems.

For each particular problem, therefore, the question is to adapt the theoretical solution arrived at in the laboratory, and it is more than likely that the final form of the covering, as well as the unit weight of the facing blocks, will differ quite noticeably from the characteristics that could be directly deduced from the general stability curves.

The case of two structures that have recently been investigated and built will show how important it is to carry out an individual investigation, and also the care that should be taken when making use of the general results.

At Crescent City, California, the structure involved was built out to sea where the water depth is about 35 ft. The structure is subjected to very high waves, of 26 ft, though a fair amount of overtopping is permissible once the waves reach a height of about 20 ft. Fig. 10 shows the section of the structure compared with the section used during the general tetrapod investigation. It will be seen that there is reasonable similarity over the lower portion but that the actual structure is cut off at a much lower elevation. This is both favorable,

in that the general stability is helped by the fact that some of the energy passes over the top of the breakwater, and also disadvantageous, in that the crest has to withstand a horizontal shearing action. The arrangement of the upper blocks and the vertical quay, which prevent scouring by the overtopping waves, solved this problem.

The 25 ton weight of the facing blocks is less than that given by the general curves. The reason for this is the low elevation of the structure, which allows an appreciable part of the energy of large waves to pass over. However, the difference in weight is small, since the two sections are not so very different in their lower portion, and, as a result of this, the transformation of waves that overtops the structure is of the same type in both cases.

On the Marine Drive Sea Wall at Bombay, India, the problem is quite different because it involves the reinforcing of an existing structure. This structure basically consists of a vertical wall, sited on a rock embankment. Over a length of several thousand feet, the structure forms the protection for the most beautiful

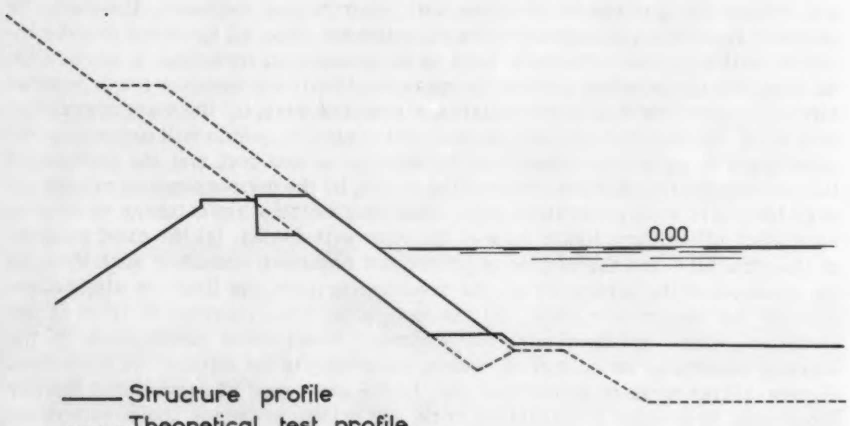

FIG. 10.—CRESCENT CITY STRUCTURE

avenue in Bombay, including an important roadway and a number of large apartment blocks. The structure is subjected to the full force of breaking monsoon waves, as well as to a fairly large rise in the sea level in the event of a storm. Any building work going above the existing crest could not be tolerated for aesthetic reasons, but overtopping had, nevertheless, to be suppressed.

Tetrapods whose size was 57 cu ft were put forward for this application since it was considered that waves would not exceed 12 ft. According to the general curves, however, 39 cu ft tetrapods should have been sufficient. The actual section of the Bombay structure, superimposed on the general test section (Fig. 11), shows the fundamental differences which exist both in the lower and upper portions.

Lighter blocks were used for Crescent City than would have been chosen if only the general results had been considered. In Bombay, however, heavier blocks were found to be necessary. If the general investigation had been used,

more money than necessary would have been spent in the first case and there would have been a risk of damage occurring in the second case.

Many other examples could be quoted, but what is most important is to know that these problems exist. They must not be overlooked or ignored.

CONCLUSIONS

Technical personnel have too few opportunities for examining an overall problem in a sufficiently disinterested way and for a sufficient length of time. Each one is bound by his particular specialized field and naturally has a tendency to only consider solutions which entirely satisfy his own particular requirements.

However, once complex questions arise, or when multiple factors are involved, it is absolutely vital that all the problems should be solved before a final solution can be considered acceptable, and even then such a solution can only be a compromise.

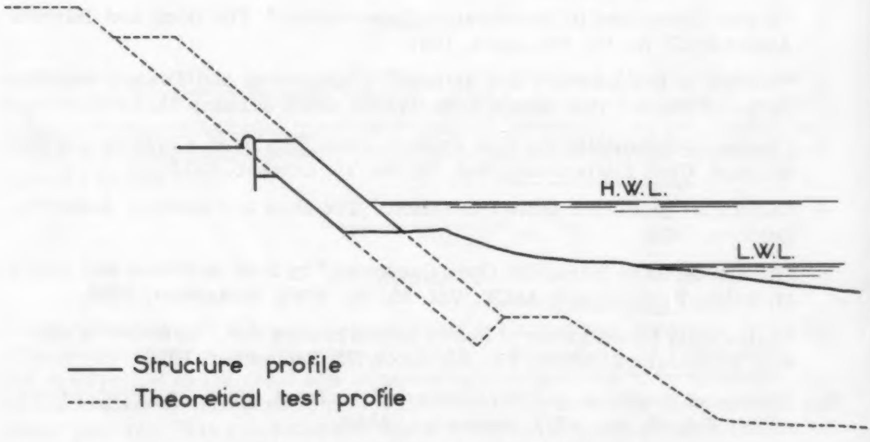

FIG. 11.—BOMBAY, INDIA SEA WALL

The development of new blocks is much more than a matter of pure hydraulic research in a laboratory. Strength of materials, corrosion, manufacturing and placing considerations, and methods of application are all factors that cannot be overlooked.

The design of the block shape, and the hydraulic developments of the block are only the first steps along the road which will end with the placing of the last block on an economically produced structure that meets the particular requirements demanded of it. To overlook one aspect of the general problem can lead to the most serious consequences. Just how large a problem the design of a new block is, and what a tremendous amount of research and development work it represents must be appreciated.

The authors have tried to draw attention to the fact that the universal panacea for coastal breakwaters does not yet exist, and it is unlikely that it will ever

do so. Every day research work results in some steps forward, but there is no general case, and, even, in this century of industrialization, the design and construction of a 4-mile-long coastal breakwater still remains a work of art.

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Proceedings of the American Society of Civil Engineers

ARKANSAS RIVER PLAN

By William Whipple,[1] F. ASCE

SYNOPSIS

Development of the Arkansas River for navigation and other purposes poses especially difficult problems on account of the quantity of sediment carried. A revised concept and plan based on relationships between slopes, depths, and widths of contracted channels results in the elimination of three dams and savings of $ 31,000,000.

GENERAL

The development of the Arkansas River for navigation purposes from the Mississippi River some 500 miles upstream to the vicinity of Tulsa, Okla., is the most recent of the great new inland navigation systems to be initiated. The entire system is estimated to cost $ 1,201,000,000, including flood control and power features. It is estimated that this system will eventually carry 13,000,000 tons of waterways traffic a year where now (1960) none exists. The Arkansas River is unique in the difficulties it presents. Although it carries a heavy load of sediment, its discharge and slope are not suitable for open channel navigation such as is under construction on the Missouri River. Locks and dams must be utilized to obtain navigable depths in the Arkansas River.

Large reservoirs now under construction will retain a large part of the sediment load, and on the main stem a combination of locks and dams and channel stabilization works will be needed. The final plan has been outlined on the basis of computed relationships between the energy slopes and depths, and the width of contracted channels. This plan represents a new concept of river develop-

Note.—Discussion open until February 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 3, September, 1960.

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ment. The result has been the elimination of at least three dams and a net savings of $31,000,000 over combinations previously contemplated.

THE OVER-ALL PLAN

The Main Control Plan of the Arkansas River will have as its principal benefit the extension of inland navigation. In addition, however, full advantage is taken of other objectives. Three main reservoirs in Oklahoma will largely regulate the Upper Arkansas and its principal tributaries, providing flood control, hydroelectric power, and regulation of flows for lockages. An essential function of these reservoirs is to retain the sediment which originates above these points. The two largest of the navigable main stem dams have sufficient head to generate hydroelectric power advantageously and will withhold further amounts of sediment. The principal characteristics of these larger projects are indicated in Table 1.

TABLE 1.—CHARACTERISTICS OF LARGER PROJECTS

Project (1)	Status (2)	Power Installation (3)	Dead Stor. & Sediment Reserve (AC F ≁) (4)	F. C. & Other Usable Storage (AC F ≁) (5)	Cost Estimate (6)
Keystone	Under Constr.	None[a]	415,000	1,464,000	$111,000,000
Oolagah	Under Constr.	None[a]	65,000	1,454,000	39,200,000
Eufaula	Under Constr.	90,000 KW	897,000	2,951,000	141,000,000
Short Mountain	Not started	110,000 KW	380,000	120,000	101,000,000
Dardanelle	Under Constr.	124,000 KW	430,000	70,000	94,600,000

[a] Power authorized, not to be initially installed.

PROBLEM AREA

The part of the plan with which this paper is primarily concerned is the river downstream from Dardanelle to the point known as Arkansas Post, where the navigation channel will leave the Arkansas River to make its exit to the Mississippi through the relatively favorable channel of the adjacent White River. This section is shown in Fig. 1.

This section of the Arkansas River will have about 190 miles of length along its improved channel. This is 27 miles less than its original length. There are no major tributaries along this reach and the minor tributaries have very little sediment content. Slope averages 0.8 ft per mile, and the variations in slope are local in character. Flood flows also are remarkably uniform, with the observed peak discharges somewhat less downstream. The river is strictly alluvial, flowing over beds of its own sediments.

The sediment content, averaging 100,000,000 tons annually, will be drastically reduced by the reservoirs. The average annual sand load of 30,000,000 tons will be reduced to an average of about 2,000,000 tons annually at Dardanelle upon closure of the Eufaula and Dardanelle Reservoirs in 1964. A quite complete channel stabilization and contraction program will be constructed, using

revetments, cut-offs, and pile and rock dikes. Physically this work closely resembles similar work on the Missouri River, with sinusoidal trace and generally parallel opposite banks.

AUTHORIZED LOCKS AND DAMS

The original plan, as revised and brought up to date periodically, contemplated a series of locks and dams superimposed over the rectified channel and interferring with it as little as possible. After earlier consideration of larger numbers of dams, eleven were planned in this portion of the river. Further savings in construction costs could, of course, have been obtained by further

FIG. 1.—PROBLEM AREA OF ARKANSAS RIVER PLAN

reducing the number of dams, but the reduction was limited by the practicable height to which pools could be raised without adversely affecting drainage and inundating or waterlogging too much agricultural land. It was realized that the channel bed would ultimately be lowered by degradation processes, but these processes are relatively slow and were not relied on to increase navigable depths.

These studies were adequate for initial purposes; however, after construction funds were appropriated for upstream reservoirs and, as more planning funds became available, an intensive system study was carried on to develop the optimum system concept of structures and basis for design. Although none of the structures involved seemed to present any particular difficulty in themselves, it was apparent that the sediment problem would profoundly affect certain design considerations. In particular, it appeared that by taking full advantage of sediment characteristics of the river, it might be possible to eliminate more of the dams, in spite of practical limitations on pool elevation.

PROGRAM OF INVESTIGATIONS

A general program of sediment observations had been conducted over a period of years so that by mid-1958, when substantial funds for general system planning were appropriated, a great deal of data were already available. Also many preliminary studies had been made. To assure an adequate scientific basis for final studies, an Arkansas River Sediment Board was organized, consisting of Messrs. L. G. Straub, H. A. Einstein, and D. C. Bondurant, to review work that had been done by district staffs and to outline further studies, data-gathering, and tests required. In 1959 a large scale movable bed model was constructed at the United States Army, Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss., and tests carried on to determine sediment carrying characteristics of a regulated channel under various conditions of contraction, and with reduced slopes. Intensive analytical and design studies were also carried on, particularly by the Little Rock District, Corps of Engineers.

BASIC PRINCIPLE

Considering a system of low-head locks and dams forming navigable pools, if the upper end of a pool is excavated deeper by dredging, but also contracted an appropriate amount, it will maintain the greater depths, with a somewhat flatter water surface slope than unaltered sections above and below, but with the same total sediment carrying capacity. By this means a dam of a given elevation may extend reliable navigable depths miles further upstream and fewer dams of limited height will extend navigation over a given reach. By August, 1958, investigations were begun into the realization of this possibility. The problem was to find ways to apply such a concept quantitatively with sufficient reliability that many million dollars could be staked upon the outcome.

SEDIMENT TRANSPORT AND DESIGN

Sediment transport is a field governed by very complex laws, which are imperfectly understood. Available literature presents only general guides that might apply to this case. Fortunately, the range of conditions to be covered for the purposes of this analysis is quite limited. Absolute results are unimportant; it is only the relative sediment carrying capacity that is involved, as between consecutive reaches, with differences of slope not over 20%, coupled with differences of channel width not over 30%. As between two adjacent sections, most other characteristics will either be identical or can be controlled by design.

From a study of all references and by checking with observed results on the principal tributaries, it was decided that the approach would be concentrated on the bed material load, or the load of sands and gravels characteristic of the bed. Secondly, it was decided that the amount of bed material load could be taken as represented by the relationship:

$$\frac{Qs}{b} = K\,(D\,S)^{X} \qquad \text{(1)}$$

in which Qs denotes the quantity of bed material load, b is the width of channel, D refers to mean depth, S is the slope, and X is an exponent. In theory, a value of 3, was determined empirically. This approach is similar in principle to the approach by H. Rouse.[2]

It was found that within the range of larger flows, the relationship of Eq. 1 on the Arkansas showed value of X generally between 2.5 and 3.0, for example, 2.8 at Little Rock. However, values were taken from experience curves, such as Fig. 2, applicable to the reach in question.

As a check a comparison was made with available "regime" theory. The origin of this theory is entirely different from that of other sediment transportation doctrine. However, since such theory is expressly concerned with design of canals of different characteristics that are designed to transport the same proportion of sediment from end to end of a network, the problem is partly analagous to what is being attempted on the Arkansas River. Direct application of equations could not be made, since regime canals have relationships

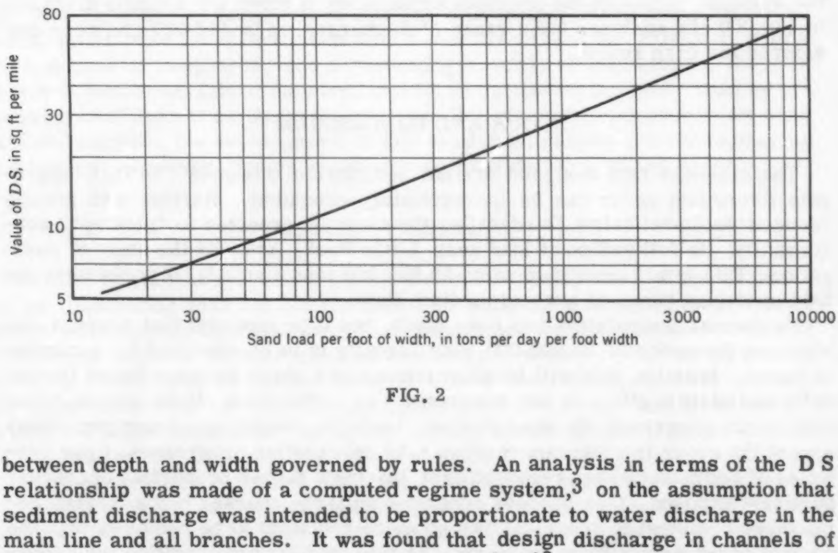


FIG. 2

between depth and width governed by rules. An analysis in terms of the D S relationship was made of a computed regime system,[3] on the assumption that sediment discharge was intended to be proportionate to water discharge in the main line and all branches. It was found that design discharge in channels of various capacities cited varied directly with $(D\ S)^3$. Therefore, it would appear that regime theory is closely consistent with the basic criterion of sediment discharge adopted for the Arkansas River.

PROCEDURES OF CHANNEL DESIGN

Design was initiated on the basis of a flow of 130,000 cfs, which was considered typical of the higher flows at which most of the sediment will move. With a normal contracted channel at a slope of 0.8 ft per mile this discharge

2 "Engineering Hydraulics," by Hunter Rouse, Fig. 15, p. 797.

3 "Regime Behaviour of Channels and Rivers," by T. Blench, Butterworths Scientific Publications, London, 1957, Table 3.1, p. 34.

will move approximately 221,000 tons of sand per day. With a given total sediment load and any other width of the channel, the corresponding sediment transport capacity per foot of width can be computed. From Fig. 2, the value of D S required to move that load is obtained. Average depths and slopes are then readily computed using normal hydraulic relationships. Navigable depths to be expected from the characteristic flow conditions in sinusoidal regulated channels were derived from mean depths on computed channels by empirical relationships.

Actual computations were somewhat more complicated than previously indicated. The characteristic trace and regulated channel actually exist only through portions of the 190 miles of river in question. Therefore, the anticipated conditions of the uniformly regulated channel had to be computed for each pool prior to computation of conditions resulting from additional contraction and excavation. Fortunately, only relative effects are important. If the total sediment carrying capacity of the more contracted sections is equal to that of the less contracted sections, actual absolute values of sediment carrying capacity are irrelevant, so long as they exceed the load delivered at the head of the system. Although the previous explanation is given for a single discharge of 130,000 cfs, analyses for a range of discharges, up to 440,000 cfs, were considered for each reach.

SCOUR AND EQUILIBRIUM

The channels thus designed are not equilibrium channels. In fact, aggregate future bed scour can be approximately computed. Starting with greatly reduced sediment below Dardanelle, the river is expected to "pick up", substantially, its full sediment load near Little Rock, Ark., at the start of navigation. This would amount to some 28,000,000 tons annually, if pools were not held up during times of low and medium flow.

No formal computation has been made, but it is apparent that average bed scour on the order of 20,000,000 tons annually is to be expected for a number of years. Initially, this will be most intense at a short distance below Dardanelle and of little effect as far downstream as Little Rock. More severe scour will occur progressively downstream, because natural armoring processes arrest the scour in upstream reaches. As degradation progresses, finer particles of bed material are removed first, leaving a higher proportion of coarser particles exposed. This process continues until the coarser bed material, being more resistant to scour, allows no more sediment to be removed than the amount coming down from upstream. At this equilibrium condition the exposed surface of the bed is characteristically covered or armored with a layer of material coarser than the average of underlying sediments.

Design of structures will have to contemplate severe degradation even below Pine Bluff, Ark., although a good many years will elapse first. Banks must of course be adequately protected, allowing for future degradation which would undermine shallow revetments. Dredging is necessary to provide adequate depths for the initiation of navigation, since degradation might otherwise take many years to reach the same depths.

Only through the armoring of the bottom will the degradation process be halted. The slope itself would never be flat enough for equilibrium otherwise. The reduction of D S values corresponding to reduced sediment availability below Dardanelle will be approximately 50%. However, a reduction of slope even

of 50% would not be sufficient to accomplish this reduction, on account of increased depths at tnat slope for a given discharge. Of course a reduction of slope as great as 50%(at high stages) cannot occur over a great length of river. Past experience elsewhere and studies of gravel occurrence in the Arkansas indicate that the equilibrium reached will be on the basis of a coarsened bed material rather than a slope proportioned to the reduced sediment load. For example, the closing off of practically all sediment flows on the Red River at Denison Dam would theoretically entail a greatly reduced equilibrium slope for many miles below. Actually degradation proceeded fairly rapidly until water levels at normal stages had been reduced at about 6 ft. The control is a gravel bar a few miles downstream. A large flood was passed in 1957 with no further degradation.

HEAD ON DAMS

One of the corollaries of the new concept is that the navigation dams must be designed to withstand a head of several feet at high stages. Originally, it had been expected to design them like the Mississippi River dams above St. Louis, which pass flood flows with only a swell head of 1 ft. However, with a given degree of roughness, and a continuing process of degradation, there is no way in which the fall of the river can be compensated for other than by increasing head loss at the dams themselves. The planned excavation below each dam will expedite the development of this head loss, despite the accompanying contraction. However, design would have to provide for it in any event.

RANGE OF RESULTS

Results of certain preliminary computations shown in Table 2 are of interest as illustrating possible alternatives. In Table 2, a comparison of Case I and IA corresponds approximately to a comparison between adjacent sections of different degrees of contraction as outlined previously. This shows that by dredging 3,600,000 cu yd and providing necessary contraction works to contract the upper section to 1,000 ft, the navigation pool of a given dam could be extended 3.7 miles upstream. However, further consideration of cases IIA and IIIA shows that the pool could be extended much further upstream, as much as 11.5 miles, by more extensive dredging, altering the basic slope of the lower section of the pool as well as the upper, more contracted section. However, for these latter cases the head on the dam itself at high stages will be increased to as much as 4.35 ft, which is greater than desirable. Moreover, the amount of dredging involved, over 19,000,000 yd per pool, is very great. Apparently, the most economic solution in this series of approximations is Solution IIIB, which resembles IIIA except that the water surface upstream from the dam is allowed to be raised an extra 0.7 ft with a consequent raising of the design bottom elevation at points above. This solution produces 6.9 miles of extra channel, and requires only 6,800,000 yd of dredging. The head on the dam is slightly less than 3 ft, but it must be noted that water surface above the dam is raised 1.5 ft. This characteristic might be unacceptable in many locations on account of flood heights.

Such results are illustrative of the range of solutions possible within which an economic design can be worked out, satisfying actual conditions at the loca-

TABLE 2.—NAVIGATION CHANNEL STUDIES[a,b,c]

Plan	Normal depth	Bed material sediment load
I	y_o = 26.0	250,000 Tons/Day
II	y_o = 26.4	225,000 Tons/Day
III	y_o = 26.85	200,000 Tons/Day

Case	Basic slope (ft/mi)	Excavation (width-feet)		Contracted width (feet)		W. S. drop at dam (feet)[a]		Total	Maximum excavation (feet)	Pool length (mile)	Excavation (cu yd)
		Upper reach	Lower reach	Upper reach	Lower reach	U/S	D/S				
I	0.80	None	None	None	None	0	0	0	0	20.00	0
II	0.75	1,370	1,370	None	None	+ 0.39	-0.67	1.06	1.06	21.33	1,326,500
III	0.70	1,370	1,370	None	None	+ 0.85	-1.44	2.29	2.29	22.86	3,072,000
I-A	0.80	600	None	1,000	None	+ 0.00	-0.47	0.47	3.00	23.75	3,600,000
II-A	0.75	600	1,370	1,000	None	+ 0.39	-1.73	2.12	5.7	27.13	9,942,000
III-A	0.70	600	1,370	1,000	None	+ 0.85	-3.50	4.35	9.2	31.56	19,508,500
III-B	0.70	600	1,370	1,150	None	+ 1.55	-1.44	2.99	5.5	26.93	6,668,000

[a] Q = 130,000 c.f.s. n = 0.0292.
[b] Normal pool assumed 28 ft above original bottom elevation at dam site.
[c] Minimum depth in 600-ft navigation channel = 12 ft.
[d] W. S. U/S from dam raised 0.7 ft above y_o = 26.85.

tion in question. Final design studies of each structure will be required before such determination.

THE GENERAL PLAN

Meanwhile, enough has been determined to enable an approximate cost estimate to be made of seven and eight-dam systems, to compare with an eleven-dam system for the reach in question. Actually the seven-dam system proved to be the cheapest in direct total cost. However, it did not provide fully adequate navigation conditions for port development at either Pine Bluff or Little Rock; it adversely affected more bottom lands. The economic effect of this loss was not fully indicated by the cost estimates and it greatly limited the flexibility of future detailed planning of individual pools and structures. Summarizing results obtained, the eleven and eight-dam systems may be compared as indicated in Table 3.

So far as operation and maintenance is concerned, there does not seem to be any pronounced difference between the two schemes. Obviously cost of operation and maintenance of the locks and dams will be proportionately less for the eight-dam plan and dike maintenance costs will be somewhat higher. It was, at first, thought that dredging maintenance costs would be greater for the eight-dam scheme, but this will not necessarily be the case. It seems likely that crossing depths will be about as predictable for the narrower channels as for the wider. The narrower deeper channels will have the same critical areas of probable shoaling as the wider shallower channels, namely, the first

TABLE 3.—COMPARISON OF ELEVEN AND EIGHT-DAM SYSTEM

	Eleven-Dam System	Eight-Dam System
Lift of dams (low flow)	10 - 14 ft	16 - 21 ft
Construction, cost, locks, and dams	$258,000,000	$197,550,000
Cost of land taken or damaged	10,050,000	8,890,000
Acres of land taken or damaged	16,180	10,950
Additional dikes, cost	$ 3,450,000	$ 10,850,000
Dredging, cubic yards	17,150,000	46,050,000
Dredging, cost	4,100,000	11,010,000
Total cost	$275,600,000	$228,300,000
Indicated Savings		$ 47,300,000

two or three crossings downstream from each dam. Under the new scheme there will be three less of such vulnerable reaches. While this study was underway, an interim decision, affecting other sections of the river, had eliminated one dam from the eleven-dam system and reduced the indicated cost by $16,000,000 on that account. Therefore, a more conservative evaluation of the savings is $31,000,000.

As far as serving its primary purpose is concerned, the eight-dam scheme has the major advantage of reducing the time of all navigation tows by eliminating three lockages. Altogether it appears as the best and soundest scheme, and it has been adopted as the basis of further detailed planning of structures concerned.

AUXILIARY STRUCTURES

The preceding analysis has rather complex implications when considered in terms of actual structures. The contraction works so far constructed on the Arkansas River are quite similar to those on the Missouri River, although the structures are further apart, leaving an open channel of 1,200 ft to 1,300 ft. Such construction results in a series of relatively stable deep pools along the concave sides of bends, joined by intermediate shallower crossings. Fig. 3 illustrates contraction works in a reach of relatively mild curvature. This reach constituted the upper third of the movable bed model.

Principal problems in planning increased contraction in given reaches concern: (a) the possibility of changed roughness coefficients when additional contraction is introduced; (b) the necessity of avoiding appreciable increases in flood heights; and (c) the feasibility of contraction effective at higher discharges, at which times there is considerable overbank flow. In addition it is necessary to consider the possible use of special structures to assure greater depths at crossings than would be maintained by normal designs.

If it can be considered that over-all roughness remains unchanged, it appears from theoretical considerations that, for a contracted channel, a high-water flow line of somewhat lesser slope than that of the normal channel will be sufficient to carry the sediment load.

This is based on the Manning Formula

$$\frac{Q}{b} = f \left(D^{5/3} S^{1/2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2a)$$

or

$$\frac{Qs}{b} = f\,(D\,S)^{2.8} \quad \text{.................} \quad (2b)$$

Dividing Eq. 2(b) by 2(a) leaves

$$\frac{Qs}{Q} = \frac{D^{2.8}\,S^{2.8}}{D^{1.7}\,S^{0.5}}$$

$$= D^{1.1}\,S^{2.3} \quad \text{....................} \quad (3)$$

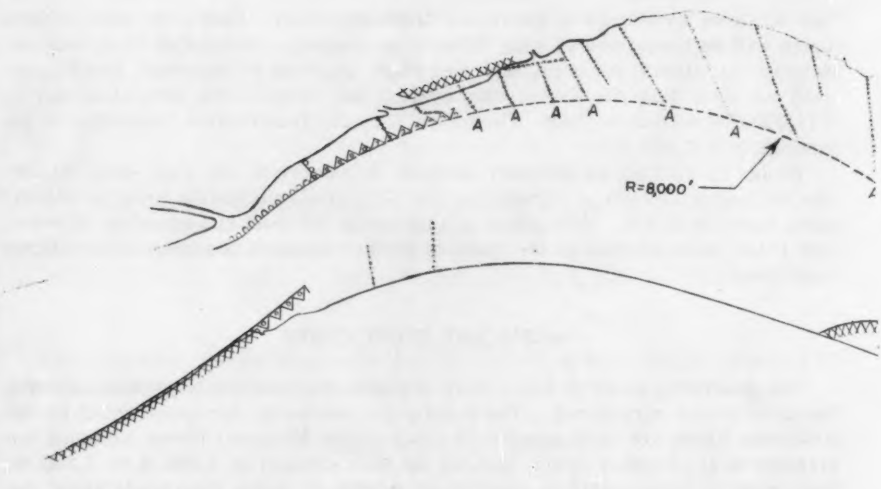


FIG. 3.—CONTRACTION WORKS IN A

The condition of continuity in unit sediment load is

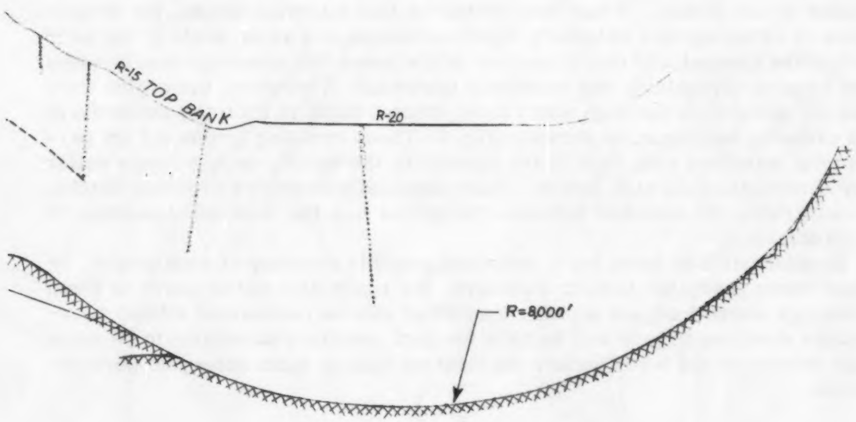
$$D_1^{1.1}\,S_1^{2.3} = D_2^{1.1}\,S_2^{2.3} \quad \text{..................} \quad (4)$$

Provided values of n are unchanged, which is implicitly assumed in the preceding, any increase in D consistent with continuity must therefore be accompanied by a reduction in slope and a consequent lowering in the flood profile.

Therefore, in cases such as previously described, contraction of overbank flows as well as channel flows can be incorporated in the program without increasing flood heights. Such overbank contraction will supplement contraction measures by means of dikes and revetments, which are most effective in regulating the lower flows. Disposition of excavated spoil so as to channel overbank flows, encouragement of willow growth, and construction of secondary

works such as Kellner Jetties in chutes and on bars not exposed to direct deep-water attack will all be considered as secondary means of contraction economical in themselves, to supplement contraction of the main channel by standard dikes and revetments. Such overbank contraction will help preserve the relative sediment-carrying capacity of contracted sections at high discharges. This is of particular importance because of the enormous loads of heavy sediments carried at such times.

The maintenance of unchanged over-all roughness coefficients in the more contracted sections involves mainly considerations of form roughness. The comparison with the natural stream need not be made, since practically all reaches downstream from Dardanelle will be contracted. With the extra contraction, the greater depth and velocity on concave sides of bends will have a



REACH OF RELATIVELY MILD CURVATURE

tendency to increase the mean form roughness on account of obstruction by contraction works, particularly if bends are sharp and of great curvature. Obviously the adjustment of planimetric layout by making cutoffs of such bends in the upper reaches of pools may be desirable where this is feasible.

A less obvious but more widely applicable means of increasing sediment-carrying capacity is to prevent the turbulence that is created by exposed ends of spur dikes on concave sides of bends, such as those indicated as "A" on Fig. 3. Such construction has long been known to create eddying, and, in fact, such construction often partially fails due to direct attack in deep water, whereas spur dikes on concave sides of bends readily silt up. The model showed a striking improvement in over-all sediment carrying capacity when such exposed ends of spur dikes were joined by longitudinal rock dikes, with the appropriate fill behind them. This result suggests that future contracted sec-

tions in the upper ends of pools may profitably incorporate longitudinal rock dikes. This action would reduce the degree of contraction otherwise required, by reducing form roughness.

The third category of special contraction works which requires further consideration is that of low-level crossing groins. Such groins are not generally used, but in one form were suggested[4] by S. Leliavsky for the purpose of increasing reliable depths obtainable at a given crossing. Leliavsky proposed a single low groin, which would, in effect, continue the curve of a concave bank well into the crossing below, thus forming a low training wall for the downstream limit of the crossing.

Such groins were constructed in the open channel model, as shown on Fig. 4, together with low spur groins delineating the opposite side of the crossing channel. Improved channel depths resulted, but an unacceptable degree of turbulence was created at high stages, and shoaling was created in the channel above the crossing.

As a second test, crossing groins were installed based on a principle suggested by the author. It has been observed that following floods, the deepest paths of crossings are relatively far downstream, at a slight angle to the main axis of the channel, and that in periods of low water the crossings tend to widen and to move irregularly and unreliably upstream. Therefore, the groins were laid out parallel to the high water flow, located so as to limit the extension of the crossing upstream, as shown in Fig. 4. These crossing groins did not perceptibly interfere with flow of the current in the model, or apparently suffer any direct attack at high stages. They materially improved crossing depths. Considerable fill resulted between the groins and the bank after passage of flood stages.

Consideration of these tests indicated possible economy of such groins, or some other generally similar structure, for application particularly to those crossings where designed depths cannot otherwise be maintained without maintenance dredging. There will be time for such auxiliary structures to be more fully developed and tested before the first navigation dams come into permanent use.

CONCLUSION

The program for extending navigation up the Arkansas (and Verdigris) Rivers to the vicinity of Tulsa, Okla., is one of the largest multiple-purpose river improvement programs ever adopted. It involves novel and complex problems from an engineering viewpoint. The initiation of construction of three of the major structures in 1955, at a time when preconstruction planning of the system as a whole was not very far advanced, represented great reliance upon the accuracy and correctness of the highly generalized pre-authorization studies and certain very preliminary approaches.

Fortunately, these conclusions require no major change in the scope and concept of the projects already under construction, except for a considerable reduction in the tailwater elevation of Dardanelle Dam, which was worked out on an interim basis prior to completion of more general studies involving the same principles. The writer and many others in the Corps of Engineers feel a profound relief that this great project, unprecedented in character, and in-

4 "An Introduction to Fluvial Hydraulics," by Serge Leliavsky, Fig. 40, p. 104.

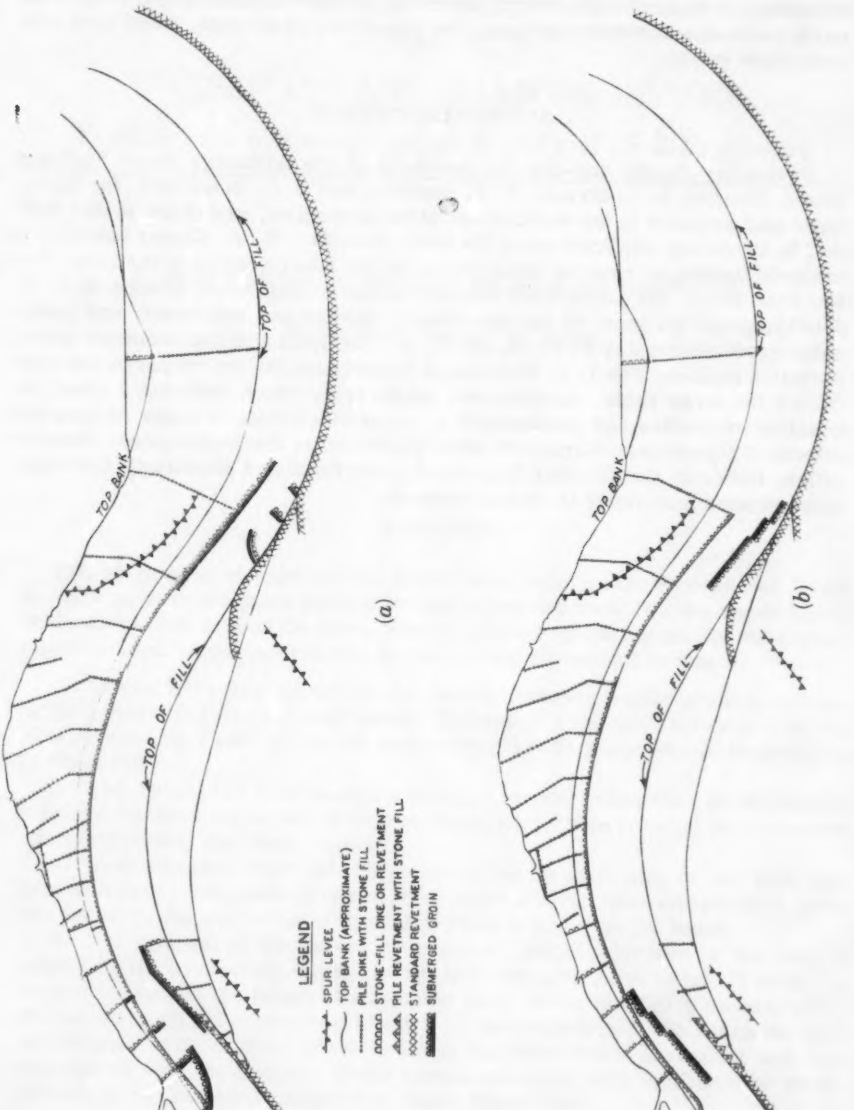


FIG. 4.—PLACING OF LOW-LEVEL CROSSING GROINS

itiated prior to the solution of all the recognized problems, is adhering to the early estimates and even indicating the possibility of savings, based upon current price levels.

ACKNOWLEDGMENTS

Particular thanks are due the members of the Arkansas River Sediment Board; Messrs. L. G. Straub, H. A. Einstein, and D. C. Bondurant, for assistance and guidance in the working out of the principles, and of the model tests and in approving applications of the basic concept. W. A. Carter assisted in decisive fashion as regards applications in the lower section of the river, below Pine Bluff. The Little Rock District, Corps of Engineers, headed by A. M. Jacoby, prepared most of the quantitative studies and estimates, and particular credit is due Jay Pyle, M. ASCE, for his hydraulic and sediment transportation studies. The U. S. Waterways Experiment Station prepared and conducted the large-scale, movable-bed model tests which afforded a check on quantitative studies and particularly a means of analysis of shape factors and effects of structures. Numerous other personnel in the Southwestern Division office, Office of the Chief of Engineers, and Tulsa and Ricksburg Districts, also assisted materially in certain aspects.

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SHARK RIVER INLET SAND BY-PASSING PROJECT

By W. Mack Angas,[1] F. ASCE

SYNOPSIS

The by-passing of sand across Shark River Inlet on the New Jersey Coast appears to be an effective method of nourishing and restoring a starved beach at the down-drift side of the Inlet. The by-passing operation, which was undertaken as a full scale experiment, is considered successful because:

1. The sand by-passed across the Inlet is coarser, better graded, and has made a beach fill that is more stable than beach fills made elsewhere on the New Jersey Coast with fine sands such as might have been obtained by dredging in Shark River.
2. The by-passed sand makes a more attractive beach than sands usually obtained by dredging in inland waters inasmuch as it is freer of objectionable shell fragments, clay balls, and silt.
3. Sand obtained from the accretion at the up-drift side of the Inlet has proved cheaper than sands obtained from other sources when comparative costs are based on the unit price of sand that stays in place on the beach.
4. The removal of the excess accretion at the up-drift side of the Inlet is benefiting the community facing the up-drift beach in three ways: It restores to utility much of the length of a "fishing pier" which was being made useless by the development of dry beach under it; the reduction of the beach berm to an optimum width of from 100 ft to 200 ft has been found desirable; and it is anticipated that the removal of the excess accretion will facilitate the maintenance of the navigation channel of Shark River Inlet.

Note.—Discussion open until February 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 3, September, 1960.

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INTRODUCTION

Shark River Inlet lies in a popular resort section of the northern New Jersey Coast, where the progressive erosion of the beaches has long been a matter of grave concern. In 1954 the Corps of Engineers, representing the Federal Government, and the Department of Conservation and Economic Development of the State of New Jersey, representing the State, completed a cooperative study of the control of the erosion of that part of the Atlantic Coast of the State lying between Sandy Hook and Barnegat Inlet which points Shark River Inlet lies.

This cooperative study led to the preparation of a report, approved by the Beach Erosion Board, whose findings and recommendations may be summarized as follows:

The 51 miles of the New Jersey Coast lying between Sandy Hook and Barnegat Inlet (Fig. 1) consist of an 11-mile stretch of barrier beach which reaches to Monmouth Beach, followed by 19 miles of headland terminating at Bay Head and finally 21 more miles of barrier beach the south end of which is Barnegat Inlet. There are littoral movements of sand northward to Sandy Hook and southward to Barnegat Inlet from a nodal point about 40 miles south of Sandy Hook and 11 miles north of Barnegat Inlet. Practically the entire 51 miles of the coastline has long been eroding progressively (Figs. 2, 3, and 4). The bulkhead shown in Fig. 2 failed to stop the progressive loss of the upland. The beach shown in Fig. 3 has gone and the upland is starting to go. During the past 50 or 60 yr this progressive erosion has been accelerated on the down-drift sides of various man-made obstructions to littoral movements of sand. These obstructions consist of jetties built at the mouths of the Manasquan and Shark Rivers to stabilize the positions of the river mouths and facilitate the maintenance of satisfactory channel depths, and groins constructed for the prevention or control of erosion.

The remedial measures recommended by the report (Fig. 5), are the restoration of starved and eroded beaches by beach filling to provide berms at least 100 ft wide and 10 ft above MLW, the construction of a few additional groins where these are considered necessary to prevent rapid reduction in the width of the widened berms by littoral movement of sand, and the regular and systematic nourishment of the beaches by establishing and periodically replenishing feeder beaches or stock piles from which sand would be distributed by previously mentioned littoral movements.

The Department of Conservation and Economic Development of the State of New Jersey, the agency responsible for the maintenance and protection of shores and beaches, is in general agreement with the recommendations of the report. Shortage of funds, however, has prevented the immediate full implementation of these recommendations. The policy that has been, and is being, followed, is to spend available funds, insofar as possible, for the accomplishment of projects compatible with the recommendations of the report and to adopt such measures as are considered appropriate when an emergency is created by a sudden localized erosion.

As a means of nourishing a starved beach and possibly stock piling sand at the down-drift side of an inlet that has a large unwanted accretion at its updrift side, sand by-passing suggested itself for several reasons. The first of these is that the State has several inlets protected by jetties, where, at the updrift side, large and unnecessary accretions have formed (Figs. 6, 7, and 8). In Fig. 6 the jetties interrupt a southwestward littoral movement of sand toward Cape May, in Fig. 7 northward littoral movement of sand is interrupted, and in Fig. 8 the starved and eroded condition of the beaches north of the inlet

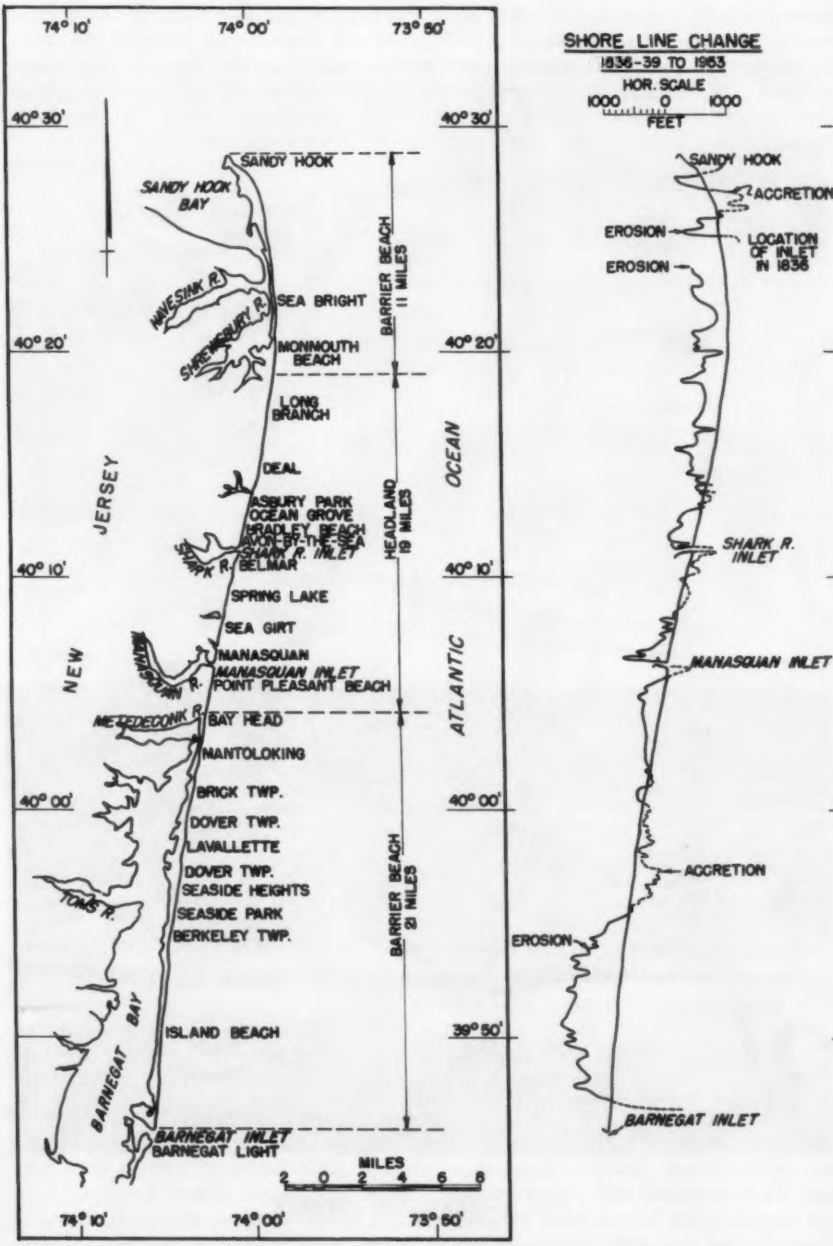

FIG. 1.—NEW JERSEY COAST SANDY HOOK TO BARNEGAT INLET

FIG. 2.—HEADLAND EROSION

FIG. 3.—HEADLAND EROSION

is evident. At all these locations there is an unfortunate lack of clean, coarse sand which could be obtained for beach filling by dredging in a nearby inland waterway. Finally, though beach filling with fine sands obtainable by suction dredging in nearby inland waters has proved reasonably effective in a number of locations, it was considered worth-while to try as an alternative the restoration of a starved down-drift beach by by-passing sand from an up-drift accretion.

SHARK RIVER INLET

Shark River Inlet (Fig. 8) was chosen as the site of the experimental by-passing operation because:

1. The beach on the down-drift or north side of Shark River Inlet is one of the most heavily eroded, highly developed ocean frontages in New Jersey.

FIG. 4.—A STARVED BEACH AND ERODING UPLAND

2. The beaches on both sides of the Inlet are publicly owned and the municipal officials and engineers of the communities owning the beaches favored the experiment and agreed to furnish necessary permissions, rights of ways and easements.

3. A bridge over the Inlet immediately behind the beach facilitated one of the by-passing methods under consideration.

The Inlet is 20 miles south of Sandy Hook and lies in the headland portion of the coast of Monmouth County. The Borough of Belmar is on the south side of the Inlet and on the north side, reading from south to north, are Avon-by-the-Sea, Bradley Beach, Ocean Grove, and Asbury Park. The beaches of all these communities are publicly owned and are heavily used during the summer season for recreational purposes. Each of the municipalities has, with financial aid from the State of New Jersey, constructed rock or timber groins along its beach front to retard loss of sand and give some measure of protection from storm damage to roads and structures on the upland. The system of groins is

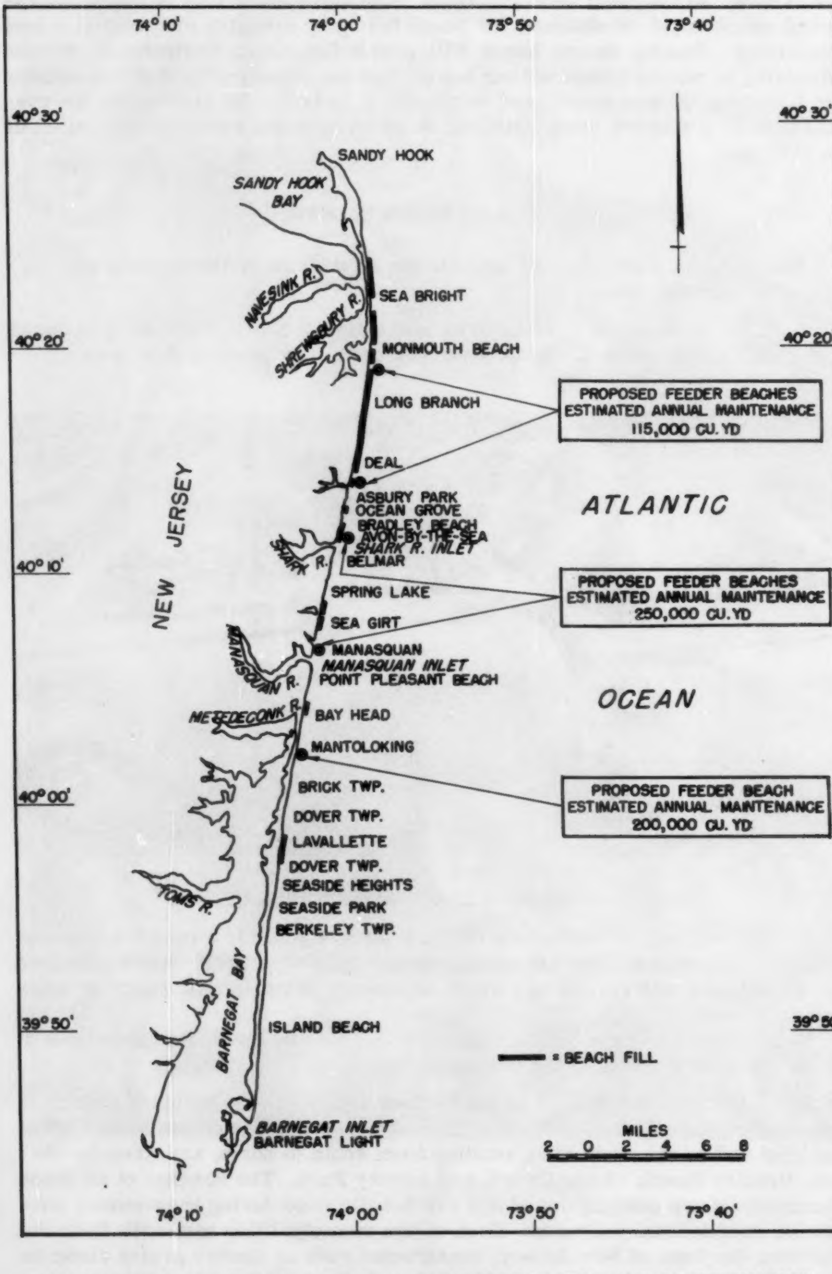

FIG. 5.—PROPOSED PLAN FOR BEACH RESTORATION AND MAINTENANCE

FIG. 6.—COLD SPRING INLET

FIG. 7.—MANASQUAN INLET

considered complete at this time and no further groin construction is contemplated within the areas under discussion.

Shark River Inlet is a federally maintained navigation inlet stabilized by jetties. The first jetties, built in 1915, were of rock and curved to the southeastward in a pair of concentric arcs. Rapid erosion of the beach and shore of Avon-by-the-Sea, at the north side of the Inlet, followed the construction of these curved jetties. Beach filling, north of the north jetty, was tried. Material used was spoil from dredging in Shark River. The fill eroded so rapidly that it did little good and was considered a failure. Reconstructing the outboard end of the south jetty with a reverse curve swinging to the northeastward also proved ineffective as a method of improving or protecting the beaches north of the Inlet.

The jetties were rebuilt and completely realigned by the State between 1948 and 1951. They now consist of a pair of parallel rock jetties 300 ft apart jutting straight out into the ocean at right angles to the shore, Figs. 8 and 9. The north jetty is 625 ft long and has a short extension or "lug" on its north side. This projection was built to protect the highly vulnerable shore alongside the

FIG. 8.—SHARK RIVER INLET

landward end of the jetty from wave action during northeasterly gales. The south jetty is longer than the north one, having a total length of 950 ft.

At the time the jetties were realigned, the Federal government dredged the Inlet and Shark River to provide a 12-ft project depth to the Belmar marina. The Federal government periodically dredges the channel between the jetties and across the bar which forms to seaward of them to a depth of 20 ft. The dredging is done by a small seagoing hopper dredge, the material being dumped at sea in deep water.

TIDES, WAVES, AND LITTORAL DRIFT

The normal range of the tide at the Inlet is 4 ft with spring ranges of 5 ft. Shark River has a small flow, is not subject to heavy flash floods, and brings down negligible quantities of sand and sediment to the beaches. Wave heights

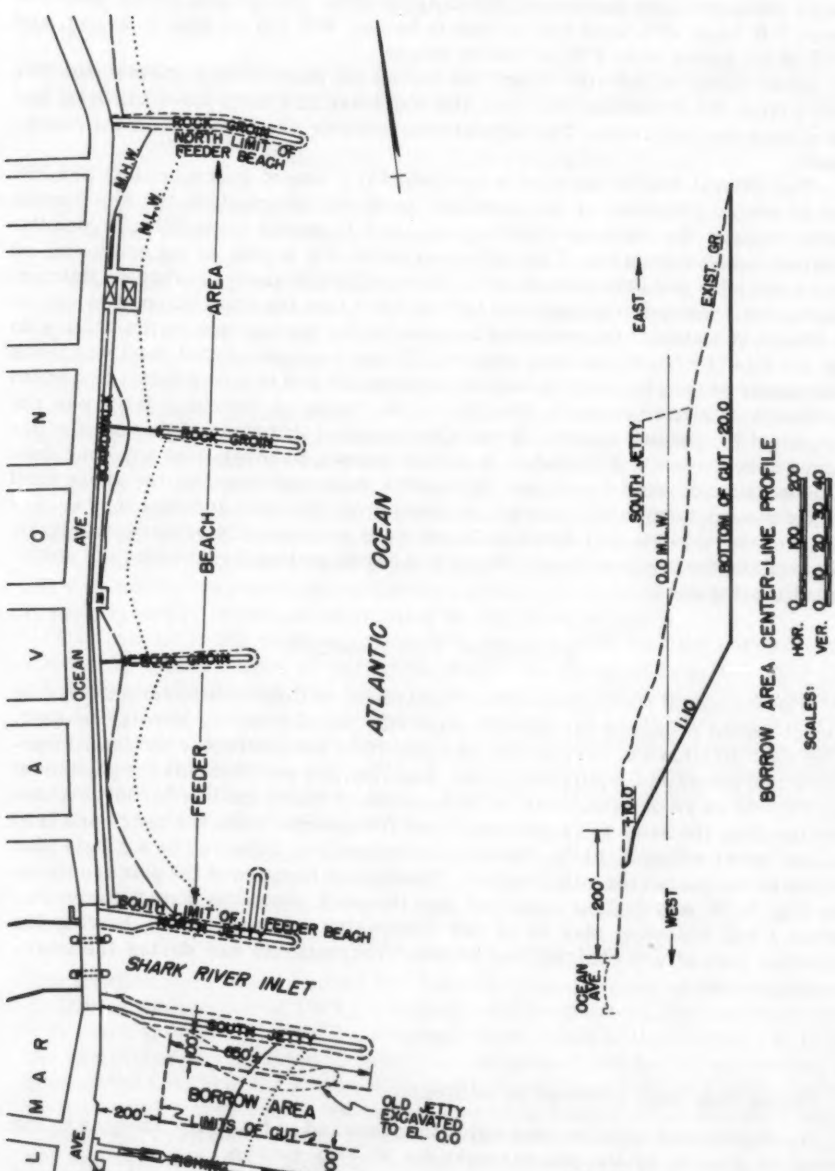

FIG. 9.—CONTRACT PLAN FOR SAND BY-PASSING PROJECT

were observed from November 1958 to April 1959. The largest waves observed were 7 ft high, 99% were 6 ft or less in height, 93% 4 ft or less in height, and 78% of all waves were 2 ft or less in height.

Observation of the wind directions during the same period showed that 19% were from the northeast, 13% from the southeast, 22% from the southwest, and 46% from the northwest. The highest wind velocity was 35 mph from the northwest.

The littoral drift at the Inlet is northward at a rate of approximately 265,000 cu yd per yr. Because of the prevalent northerly littoral drift, the 950-ft south jetty trapped the northward moving sand and formed a large fillet beyond the normal beach alignment. This fillet was some 400 ft wide at the south side of the south jetty and extended about 1,400 ft southward along the shore of Belmar. It was the excess beach material in this fillet that the State planned to use as a source of material for restoring and nourishing the beaches on the north side of the Inlet by the by-passing project. It was recognized that sand was being by-passed across the Inlet by natural processes[2] and believed that a bar which extended in a northeasterly direction from the tip of the south jetty was the principal by-passing agency. It was also apparent that some of this sand nourished beaches north of the Inlet. It did not appear, however, that any considerable quantity of sand by-passed by natural processes reached the shore until it had moved northward a number of miles from the Inlet and that the beaches of Avon-by-the-Sea and Bradley Beach were consequently being destructively starved and the beach at Ocean Grove was hardly getting its fair share of northward moving sand.

PLANNING THE PROJECT

The Bureau of Navigation of the Department of Conservation and Economic Development prepared the specification and plan for the by-passing project. The specification was very simple requiring that the contractor furnish all construction materials, equipment, labor, supplies, and facilities for the placement of 250,000 cu yd of sand, more or less, as beach fill on the beachfront of Avon-by-the-Sea, the sand to be excavated and transported from the beach and land under water adjacent to the Borough of Belmar, as indicated on a single plan which accompanied the specification. The salient features of the plan are shown on Fig. 9. It was further specified that the work should be done between October 1 and following May 31 of two successive winter seasons, leaving the beaches free of any obstructions to their recreational use during the intervening summer.

BIDS

Three bids were received as follows:

1. A proposal to do the excavation by crane and to transport the sand to the beaches of Avon-by-the-Sea in trucks for $0.88 per cu yd.
2. A proposal to excavate the sand by dragline, place it in a hopper, and transport it hydraulically to the beach fill by means of a dredge pump and pipe line. The price quoted by this bidder was $1.30 per cu yd.

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2 "Natural By-Passing of Sand at Coastal Inlets," by Per Bruun and F. Gerristsen, Proceedings, ASCE, Vol. 85, No. WW 4, December, 1959, pp. 75-107.

3. A proposal to do the work with a hydraulic dredge and pipe line for a price of $1.42 per cu yd.

The contract was awarded on the first proposal at the bid price of $0.88 per cu yd for excavating the sand with a crane and bucket and transporting it by truck.

EXECUTION OF THE PROJECT

During late September and October 1958 the contractor built a trestle in the borrow area reaching from Ocean Avenue to a little beyond the low water mark. The trestle was about 425 ft long. He also built three 200-ft trestles north of the Inlet from which to dump sand on the beaches to be filled. The locations of these trestles are shown on Fig. 9. Two of the trestles were in Avon-by-the-Sea and the third on the line dividing Avon-by-the-Sea from Bradley Beach.

On about November 1, the contractor commenced excavating sand from the borrow area with a crane and 2-1/2 cu yd clam shell bucket. A bulldozer was used on the beach to push sand to the crane (Fig. 10). Transportation of the sand to Avon-by-the-Sea was done with two 20-cu-yd trucks and one 10-yd truck. The trestles on which the sand was delivered to the beaches north of the Inlet reached a little beyond low water (Fig. 11), and the contractor proposed to leave the distribution and spreading of the sand from the dumping points at the ends of the trestles to wave action. This would be permitted if wave action proved effective. It did in fact prove highly effective and no spreading of the fill with bulldozers proved necessary during the first winter's operation, though it was necessary to extend the trestles as the beach widened.

The rate at which sand was moved by littoral drift into the borrow area during the winter season of northwest winds was disappointing. On April 17, 1959, the supply of sand that could be reached by the contractor's equipment was insufficient to justify further work at the time and excavation was accordingly stopped. The amount of sand that had been placed on the beaches north of the Inlet was, however, considerable and had effected a great improvement in the condition of the beaches (Figs. 12 and 13). To be specific, the amounts of fill placed from the trestles were:

Trestle 1 —	38,000 cu yd, truck measure.
Trestle 2 —	48,000 cu yd, truck measure.
Trestle 3 —	51,000 cu yd, truck measure.
Total	137,000 cu yd, truck measure.

The trestles were then removed and beaches cleaned up for summer use.

During the summer of 1959 the normal northward littoral movement of sand took place and the excavated borrow area filled rapidly. On October 15, 1959, the contractor was directed to reactivate the project and he at once started to replace the trestle in the borrow area and to reconstruct trestle 1 and 2. Trestle 3 was not reconstructed but two new trestles, 4 and 5 (Fig. 15), were built. Trestle 4 is in Bradley Beach between the northernmost groin in Avon-by-the-Sea and the first one in Bradley Beach. Trestle 5 is north of the first on the Bradley Beach groins. Trestles 4 and 5 do not reach as far into the water as do trestles 1 and 2, and it has been necessary to spread some of the sand dumped from them by means of a bulldozer. Excavation and placement of sand during the second season started on October 22, 1959 and to date 88,000 cu yd have been by-passed this winter or 225,000 cu yd since the project was started in September, 1958. About 25,000 cu yd, therefore, remain to be moved across

FIG. 10.—EXCAVATING SAND FROM THE BORROW AREA

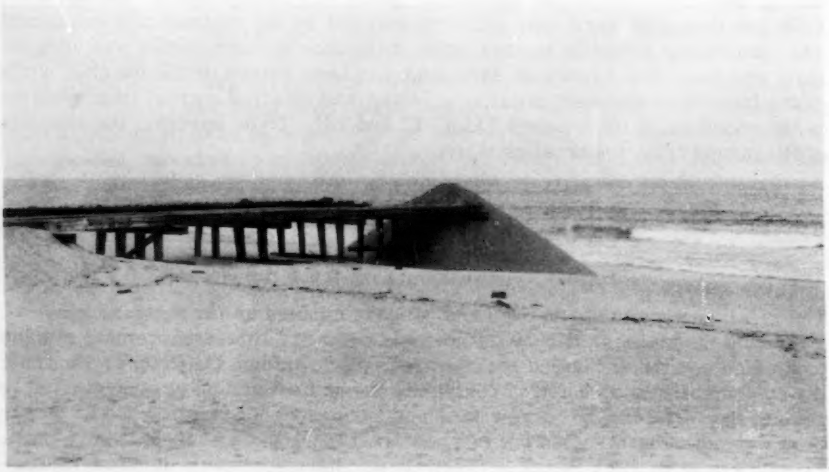

FIG. 11.—SAND DUMPED ON THE BEACH TO BE DISTRIBUTED BY WAVE ACTION AT HIGH WATER

FIG. 12.—CONDITION OF THE BEACHES AT THE COMMENCEMENT OF BY-PASSING IN 1958.

FIG. 13.—CONDITION OF THE BEACHES AT THE END OF THE FIRST WINTER'S WORK

the inlet. This will undoubtedly be accomplished by early April. Most of the sand is now being delivered over trestles 4 and 5 as the beaches initially fed through trestles 1, 2, and 3 are in excellent condition and did not erode seriously during the past summer (Fig. 13).

It is anticipated that the entire 250,000 cu yd to be moved under the contract will have been by-passed across the Inlet before the middle of April and the project will then be complete.

RESULTS

When sand by-passing by the method used at Shark River Inlet was proposed, four important questions were raised. These were:

1. Could the contractor maintain a satisfactory schedule of excavation and placement by the methods he proposed to use?
2. Would the removal of sand in the planned quantity damage Belmar Beach?

FIG. 14.—CONDITION OF THE BEACHES BEFORE COMMENCEMENT OF THE SECOND WINTER'S WORK

3. Would littoral drift bring material to the borrow area at a rate sufficient to maintain the planned work schedule?
4. Could sand be satisfactorily distributed and spread on the filled beaches if it were dumped onto them from trestles?

The results accomplished show:

a. The contractor has maintained a satisfactory schedule of placement inasmuch as he excavated and placed 137,000 cu yd during the first winter he worked. This is more than half the 250,000 cu yd he was to move during two winters.

b. Aerial photographs and maps (Figs. 12, 13, 14, 15, 16, and 17), show that the alignment of the Belmar beach has been maintained in a very satisfactory way. The portion removed was only that part of the fillet south jetty which was considered excess material.

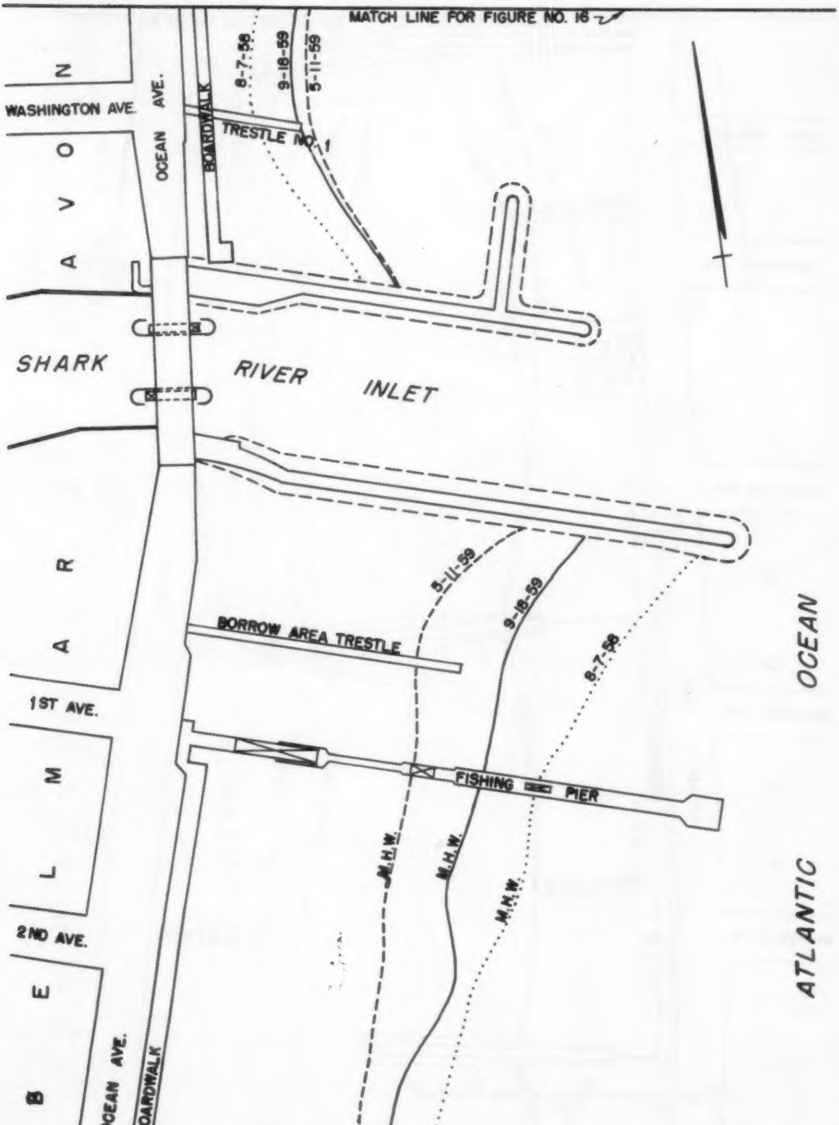

FIG. 15.—PLAN SHOWING RESULT OF SAND BY-PASSING PROJECT

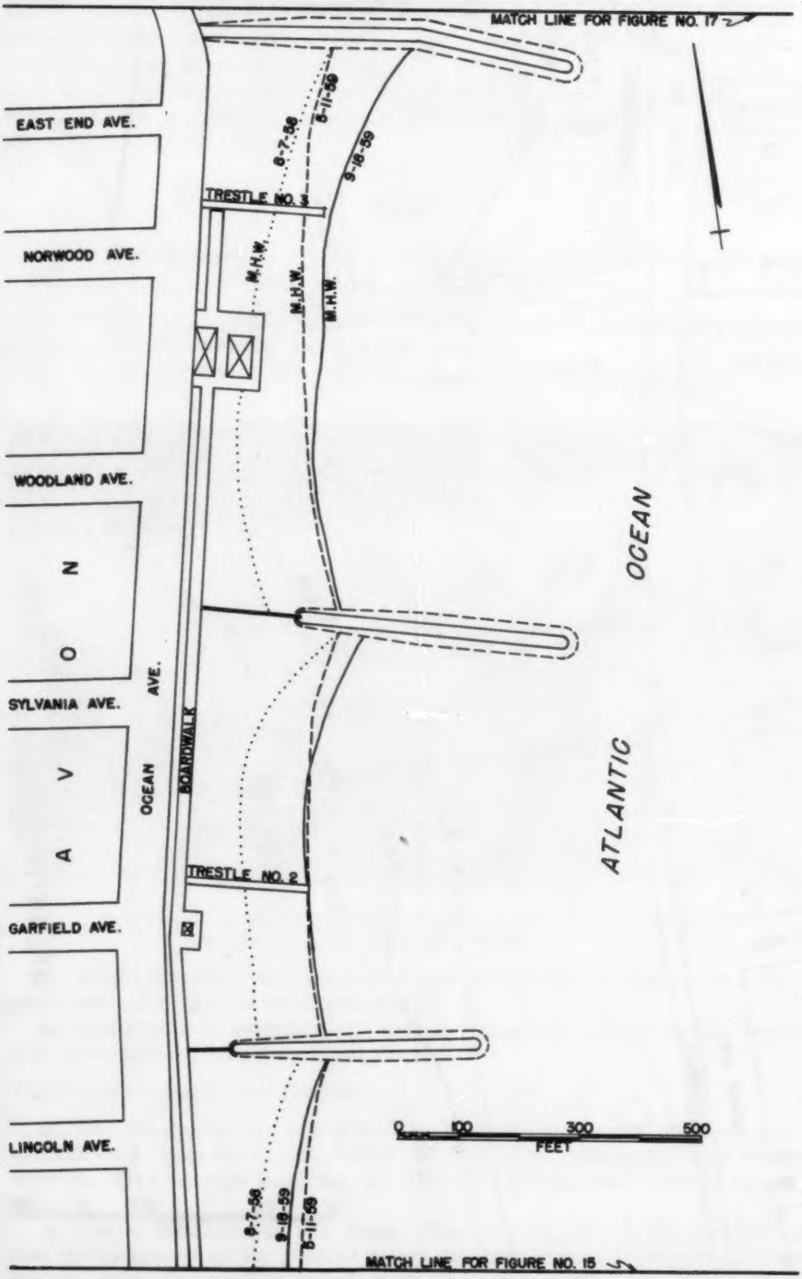

FIG. 16.—PLAN SHOWING RESULT OF SAND BY-PASSING PROJECT

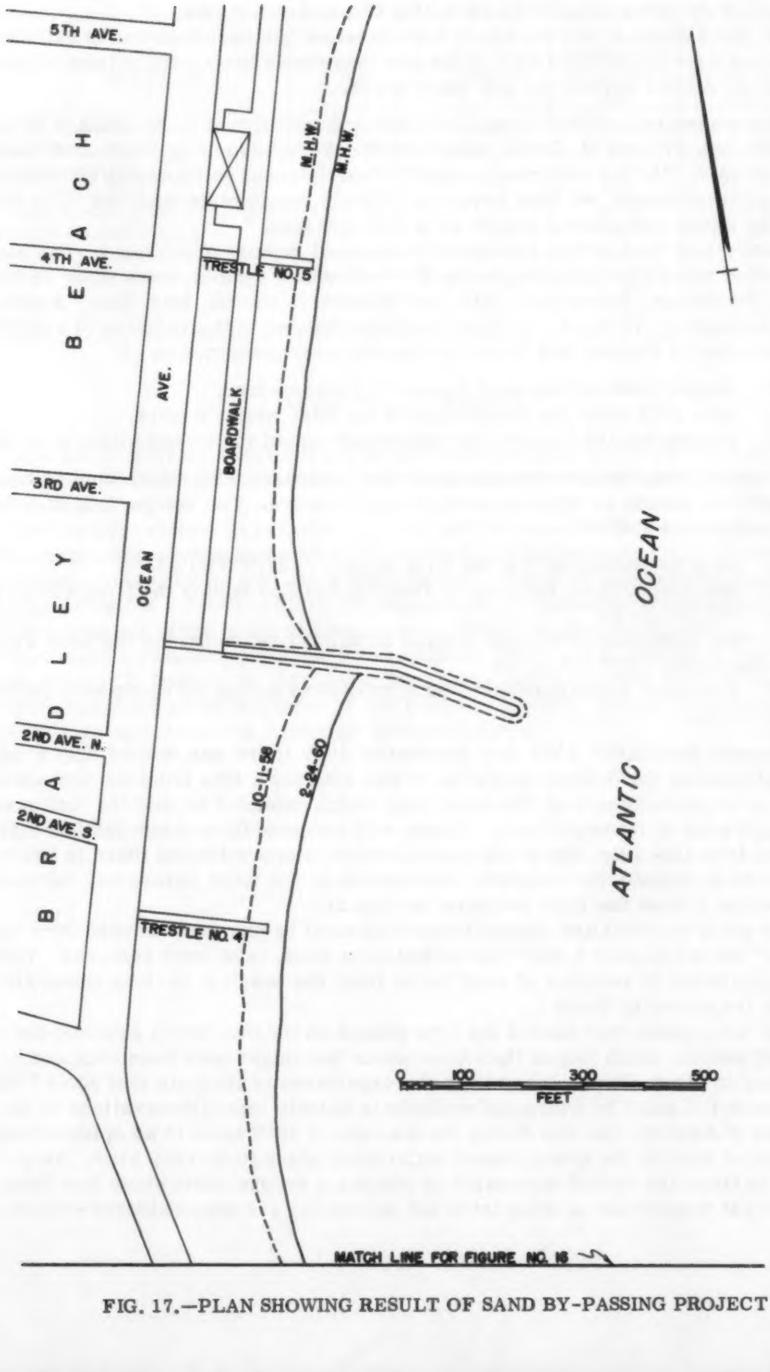

FIG. 17.—PLAN SHOWING RESULT OF SAND BY-PASSING PROJECT

c. Material has been brought into the borrow area at a sufficient rate to permit by-passing 250,000 cu yd during two winter seasons.

d. Sand dumped onto the beach from trestles has been distributed satisfactorily by wave action when the trestles are reasonably closely spaced and reached out to or a little beyond the low water mark.

In commenting on the present character and condition of the beaches of Avon-by-the-Sea, Francis B. Cogan, Assistant Chief Engineer of the Bureau of Navigation, said: "In our experience with the nourishment and pumping of beaches by hydraulic means, we have never had results comparable with the distribution by waves and natural forces as in this instance."

Surveys of the beaches and ocean frontages of Belmar and Avon-by-the-Sea, including soundings extending to the 25-ft off-shore contour, were made in August, November, December, 1958, and February, March, May, June, August, and September, 1959. From these surveys changes in the volumes of sand on the beaches of Belmar and Avon-by-the-Sea were computed as of:

(1) August 1958 before sand by-passing commenced.
(2) May 1959 after the completion of the first winter's work.
(3) September 1959 before the commencement of the second winter's work.

The resulting estimated volumes were then compared with the volume of sand actually by-passed as determined by truck measure. The comparison may be summarized as follows:

(a) Sand by-passed during the first winter: 137,000 cu yd.
(b) Net accretion on beaches of Avon-by-the-Sea during the first winter's work: 150,000 cu yd.
(c) Net reduction of volume of sand on Belmar beach during the first winter's operation: 129,000 cu yd.
(d) Volume of sand returned to the borrow area during the summer of 1959: 89,000 cu yd.

Between September 1958 and September 1959 there was accordingly a net loss of sand on the Belmar beach but it was entirely a loss from the unwanted fillet or accretion south of the south jetty which extended beyond the optimum outboard edge of the beach berm. There will undoubtedly be some further loss of sand from this fillet due to the second winter's operation but there is every reason to anticipate the complete restoration of the fillet before any further by-passing across the Inlet becomes necessary.

The sand removed and placed is medium sand having been worked over by littoral forces in such a way that undesirable fines have been removed. The characteristics of samples of sand taken from the beach at various times and places are shown in Table 1.

It is anticipated that sand of the type placed on the beaches of Avon-by-the-Sea will remain much longer than finer sands that might have been obtained by dredging in Shark River. It has been the experience of the State that about 25% of a beach fill made by hydraulic methods is quickly lost. Observations of the beaches of Avon-by-the-Sea during the summer of 1959 leads to an opinion that the rate of loss of the newly placed material is going to be very slow. An attempt to trace the rate of movement by placing a recognizable stone dust from a quarry at Somerville is being tried but the results are not considered encouraging.

TABLE 1.—SAND CHARACTERISTICS

Sample (1)	Date (2)	Median Diameter (3)	Coefficient of sorting (4)	Skewness (5)
Avon Trestle No. 2	Sept. 1958	.43	1.41	1.08
Avon Trestle No. 2	Oct. 1959	.72	1.69	0.97
Avon Trestle No. 1	Oct. 1959	.62	1.49	0.97
Avon Trestle No. 3	Oct. 1959	.51	1.67	1.17
Bradley Beach Trestle No. 4	Oct. 1959	.37	1.21	1.04
Belmar - Borrow Area	Jan. 1959	.98	1.53	1.00
Belmar - Borrow Area	Oct. 1959	1.24	1.46	0.96
Belmar - 2nd Avenue	Oct. 1959	1.20	1.39	0.92
Belmar - 4th Avenue	Oct. 1959	0.56	1.23	1.03
Belmar - 6th Avenue	Oct. 1959	0.50	1.27	0.89

ACKNOWLEDGEMENTS

The success of the project must be attributed largely to the care and attention to its direction given personally by Thomas Procter, the contractor, and F. B. Cogan.

The author wishes to acknowledge his indebtedness to those who gave the encouragement, advice, and assistance which made the preparation of this paper possible. Among those who were most helpful should be mentioned Commissioner Salvatore A. Bontempo, of the Department of Conservation and Economic Development of the State of New Jersey; Director Kenneth Kreveling, of the Department's Planning Division; Peter Gannon, Chief of the Bureau of Navigation; James K. Rankin, M. ASCE, the Bureau's Chief Engineer; and Francis B. Cogan, Assistant Chief Engineer of the Bureau. Finally, Harold Shinn must be thanked for preparing the drawings used as figures.

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PNEUMATIC BARRIER AGAINST SALT WATER INTRUSION

Ian Larsen[1]

SYNOPSIS

In rivers subject to salt water intrusion, bubbles of air from perforated pipes on the river bottom will create an upward flow of salt water and thus a mixing between the fresh and salt water layers. When the upward flow reaches the absolute maximum of salt water discharge the salt water will be unable to penetrate the bubble "curtains." Such installations may provide the answer to the salt water problem in many rivers.

INTRODUCTION

The difference in density is known to be the cause of salt water intrusion in estuaries. In this connexion, three types of intrusion are recognized:

1. Estuaries with a salt water wedge, that is, the salt water, sharply separated from the fresh water, intrudes along the river bed in the shape of a wedge.
2. Well mixed estuaries, which are generally characterized by vertical isohalines.
3. Partly mixed estuaries, that form the transition between the two types already mentioned.

Note.—Discussion open until February 1, 1961. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 3, September, 1960.

[1] Research Engr., Coastal Engrg. Lab., Tech. Univ. of Denmark, Copenhagen, Denmark.

The mixing conditions in an estuary seem to be determined chiefly by the ratio between the fresh water discharge in a tidal period and the tidal prism.[2]

In many places in the world salt water pollution creates serious problems with regard to both agriculture and water supplies. Therefore, it is valuable to develop methods of hindering this intrusion.

Training walls, sills, and other control structures have been used in the fight against the salt water, but their effectiveness is limited because of navigational requirements. It is believed that the pneumatic salt water barrier, either alone, or in combination with the previously mentioned structures, may become a valuable means for the control of salt water intrusion.

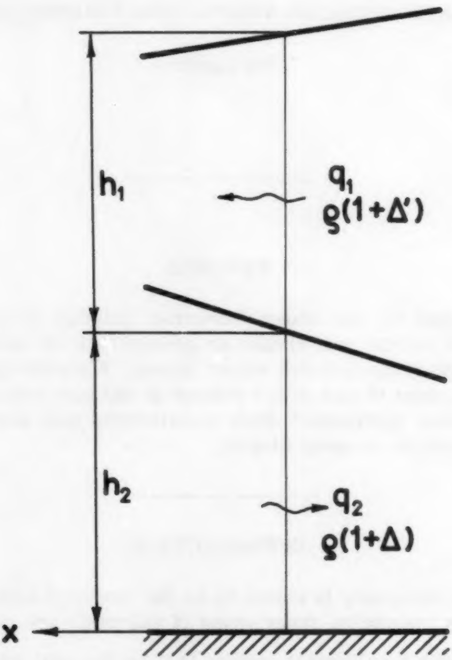

FIG. 1.—NOTATIONS OF TWO-LAYER FLOW AT RIVER MOUTH

The fundamental principle of the pneumatic salt water barrier is that the bubbles of air emitted from perforated pipes placed on the river bed will create an upward flow of salt water that will be mixed with the upper layer of outgoing fresh water. In partly mixed estuaries and estuaries with a salt water wedge, the natural velocities in the lower layer of salt water are small. Application of a pneumatic barrier will change the hydraulic conditions of the

[2] "Some Effects of Upland Discharge on Estuarine Hydraulics," by Henry B. Simmons, ASCE, Proc. Paper 792, Sept. 1955.

barrier downstream and at the river mouth especially. Therefore, as an introduction to the description of the pneumatic barrier it is necessary to consider the hydraulic conditions at the river mouth.

GENERAL THEORY OF TWO-LAYER DENSITY FLOW AT RIVER MOUTHS

At a river mouth or at any other location where an abrupt widening of the cross-section occurs, there is a sudden decrease in the thickness of the upper layer.

For the stability conditions of this hydraulic jump long, internal waves are stationary. Neglecting the shear stresses at the bottom and at the interface, as well as the mixture of layers in the vicinity of the jump, the equilibrium equations for the upper and lower layers of densities ($\rho(1 + \Delta')$ and $\rho(1 + \Delta)$, respectively) are in accord with the notations given in Fig. 1.

$$\rho\,(1 + \Delta')\,\frac{d}{dx}\left(\frac{q_1^2}{h_1}\right) = -\rho\,(1 + \Delta')\,g\,h_1\,\frac{d}{dx}\left(h_1 + h_2\right) \;\ldots\ldots\; (1)$$

and

$$\rho\,(1 + \Delta)\,\frac{d}{dx}\left(\frac{q_2^2}{h_2}\right) = -\,\rho\,(1 + \Delta')\,g\,h_2\,\frac{d}{dx}\left(h_1 + h_2\right) - \rho\,(\Delta - \Delta')\,g\,h_2\,\frac{dh_2}{dx} \;\ldots\; (2)$$

in which q_1 and q_2 denote the discharges per unit width of flow.

If the fresh water discharge of the river is called q_o, the continuity equation of the water gives:

$$q_1 = q_o + q_2 \;\ldots\ldots\ldots\ldots\ldots\; (3)$$

The continuity equation for the dissolved material, that creates the difference in density between the layers, may be expressed as follows:

$$\Delta q_2 = \Delta' q_1 \;\ldots\ldots\ldots\ldots\ldots\; (4)$$

If ψ denotes the ratio between the discharge of q_2 of the lower layer and the fresh water discharge q_o, Eq. 3 can be converted to:

$$q_1 = (1 + \psi)\,q_o \;\ldots\ldots\ldots\ldots\ldots\; (5)$$

which when inserted in Eq. 4, gives:

$$\frac{\Delta'}{\Delta} = \frac{\psi}{1 + \psi} \;\ldots\ldots\ldots\ldots\ldots\; (6)$$

Eqs. 1 and 2 can be made dimensionless by means of the parameters:

$$\xi = \frac{x}{h_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

$$\eta_1 = \frac{h_1}{h_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

$$\eta_2 = \frac{h_2}{h_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

and

$$F^{-2} = \frac{g\, h_o^3}{q_o^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

in which h_o is the depth of the channel and F is the Froude number of the river. Thus, Eqs. 1 and 2 become

$$(1 + \Delta') \frac{d}{d\xi}\left[\frac{(1+\psi)^2}{\eta_1}\right] = -(1 + \Delta')\, F^{-2}\, \eta_1 \frac{d}{d\xi}\left(\eta_1 + \eta_2\right) \quad \ldots (11)$$

and

$$(1 + \Delta) \frac{d}{d\xi}\left(\frac{\psi^2}{\eta_2}\right) = -(1 + \Delta') F^{-2}\, \eta_2 \frac{d}{d\xi}\left(\eta_1 + \eta_2\right) - (\Delta - \Delta')\, F^{-2}\, \eta_2 \frac{d}{d\xi}\eta_2 \,.\,. (12)$$

By differentiation of the individual terms and regrouping, the following is obtained:

$$\left[\frac{(1+\psi)^2}{\eta_1^2} - \eta_1 F^{-2}\right] \frac{d\eta_1}{d\xi} = \eta_1 F^{-2} \frac{d\eta_2}{d\xi} \quad \ldots\ldots\ldots\ldots (13)$$

and

$$(1 + \Delta')\, \eta_2\, F^{-2} \frac{d\eta_1}{d\xi} = (1 + \Delta)\left(\frac{\psi^2}{\eta_2^2} - \eta_2 F^{-2}\right) \frac{d\,\eta_2}{d\,\xi} \quad \ldots\ldots (14)$$

The previously mentioned stability condition of the hydraulic jump involves Eqs. 13 and 14 being satisfied by any value of $\frac{d\eta_1}{d\xi}$ and $\frac{d\eta_2}{d\xi}$. In other words, the

equations must be identical. Hence.

$$(1+\Delta')\,\eta_2\,\eta_1\,F^{-2} = (1+\Delta)\left(\frac{\psi^2}{\eta_2^2} - \eta_2\,F^{-2}\right)\left[\frac{(1+\psi)^2}{\eta_1^2}\,F^2 - \eta_1\right] \quad ..(15)$$

or

$$\frac{\Delta-\Delta'}{1+\Delta}\,\eta_1\,\eta_2\,F^{-2} = \frac{\eta_1}{\eta_2^2}\psi^2 + \frac{\eta_2}{\eta_1^2}(1+\psi)^2 - \frac{\psi^2(1+\psi)^2}{\eta_2^2\,\eta_1^2}\,F^2 \quad ...(15a)$$

Inserting Δ' from Eq. 6 results in

$$\frac{\Delta}{1+\Delta}\,\frac{1}{1+\psi}\,\eta_1^3\,\eta_2^3\,F^{-2} = \psi^2\,\eta_1^3 + (1+\psi)^2\,\eta_2^3 - \psi^2(1+\psi)^2\,F^2 \quad ..(15b)$$

In Eq. 15b the usual densimetric Froude Number F_Δ as defined by Eq. 16 may be introduced.

$$F_\Delta^{-2} = \frac{\Delta}{1+\Delta}\,F^{-2} \quad ....(16)$$

Since F^2 is normally small and

$$\eta_1 + \eta_2 \sim 1 \quad ....(17)$$

Eq. 15b may be approximated by:

$$\phi\,(\eta_1\,\psi) = \psi^2\,\eta_1^3 + (1+\psi)^2\,(1-\eta_1)^3 - \frac{1}{1+\psi}\,\eta_1^3\,(1-\eta_1)^3\,F_\Delta^{-2} = 0 \quad ..(18)$$

Now, η_1 lays in the interval between 0 and 1. Further analysis of Eq. 18 shows that for each value of F_Δ there exists a certain value of ψ, ψ_m, so that for $0 \leq \psi < \psi_m$ Eq. 18 has two solutions for η_1, the larger solution corresponding to an unstable condition. Values of $\psi > \psi_m$ will result in Eq. 18 having no roots. For $\psi = \psi_m$, Eq. 18 has a double root, η_m. For $\psi = o$, Eq. 18 has, apart from the unusable root $\eta_1 = 1$, the root:

$$\eta_1 = \eta_\kappa = F_\Delta^{2/3} \quad ....(19)$$

In the case of a stationary salt water wedge, in which the natural mixing between the layers causes only low velocities in the salt water layer, η_κ denotes the dimensionless thickness of the layer of fresh water at the river mouth. Fig. 2 shows the roots of Eq. 18 for various values of η_κ, while the correlation between η_κ and the densimetric Froude Number F_Δ is given in Fig. 3.

Eq. 18 yields the double root, η_m, when

$$\left(\frac{\delta\phi}{\delta\eta}\right)_{\eta=\eta_m} = 3\left(\psi_m^2 \eta_m^2 - \left(1+\psi_m\right)^2 \left(1-\eta_m\right)^2 - \frac{1}{1+\psi_m}\eta_m^2 \left(1-\eta_m\right)^2\right.$$

$$\left.\left(1 - 2\eta_m\right) F_\Delta^{-2}\right) = o \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

In this case Eq. 18 gives:

$$\psi_m^2 \eta_m^3 + \left(1+\psi_m\right)^2 \left(1-\eta_m\right)^3 - \frac{1}{1+\psi_m}\eta_m^3 \left(1-\eta_m\right)^3 F_\Delta^{-2} = o \quad \ldots (21)$$

Eliminating F_Δ from Eqs. 20 and 21 we find:

$$\eta_m = \frac{1}{1+\sqrt{\frac{\psi_m}{1+\psi_m}}} \quad \ldots\ldots\ldots\ldots\ldots (22)$$

The relation between ψ_m and F_Δ may be found by inserting Eq. 22 into Eq. 18, which yields

$$F_\Delta^{-2} = \left(1+\psi_m\right)\left(\sqrt{1+\psi_m}+\sqrt{\psi_m}\right)^4 \quad \ldots\ldots\ldots\ldots (23)$$

Fig. 3 shows ψ_m, η_m and η_x versus F_Δ^{-2}.

The quantity of salt water necessary to create the double-root condition can be expressed by ζ, and defined by

$$\zeta = \frac{q_{2,m}}{\sqrt{\Delta g h_o^3}} = \frac{\psi_m q_o}{\sqrt{\Delta g h_o^3}} \sim \psi_m F_\Delta = \frac{\psi_m}{\sqrt{1+\psi_m}\left(\sqrt{1+\psi_m}+\sqrt{\psi_m}\right)^2} \quad \ldots (24)$$

In Fig. 3 ζ is shown as a function of F_Δ^{-2}. The term ζ has a maximum value of

$$\zeta_{max} = 0.122 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

that corresponds to

$$q_{2,max} = 0.122\sqrt{\Delta g h_o^3} \quad \ldots\ldots\ldots\ldots\ldots (26)$$

BASIC PRINCIPLE OF THE SALT WATER BARRIER

The stationary salt water wedge has been previously considered, in which the flow in the salt water layer was moderate in relation to the flow in the upper layer. The dimensionless thickness of the upper layer at the river mouth was given by Eq. 19.

Now assume that the vertical mixing between the two layers is intensified. The upper layer becomes increasingly saline, the inward and outward currents are increased simultaneously, and the thicknesses of the two layers of the river mouth can be determined by Eq. 18.

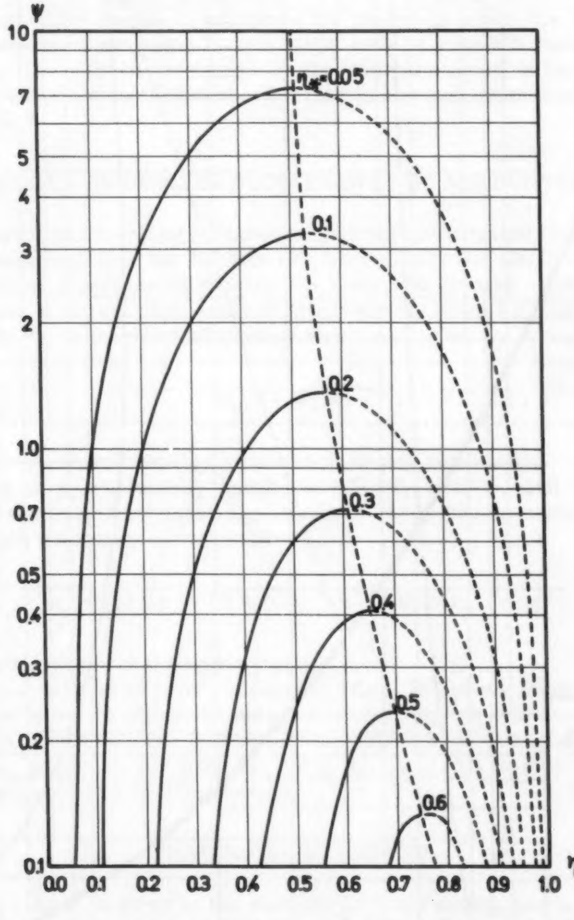

FIG. 2.—THE DIMENSIONLESS THICKNESS OF THE FRESH WATER LAYER η_1 AT THE RIVER MOUTH.

However, it is only possible by this means to increase the flows in the upper and lower layers to a limit corresponding to ψ_m in Eq. 23. Further attempts to intensify the mixing will have no result. This fact has been proved in experiments by Stommel and Farmer.[3]

[3] "Control of Salinity in an Estuary by a Transition," by Henry Stommel and Harlow G. Farmer, Journ. Mar. Res., Vol. XII, 1953, pp. 13-20.

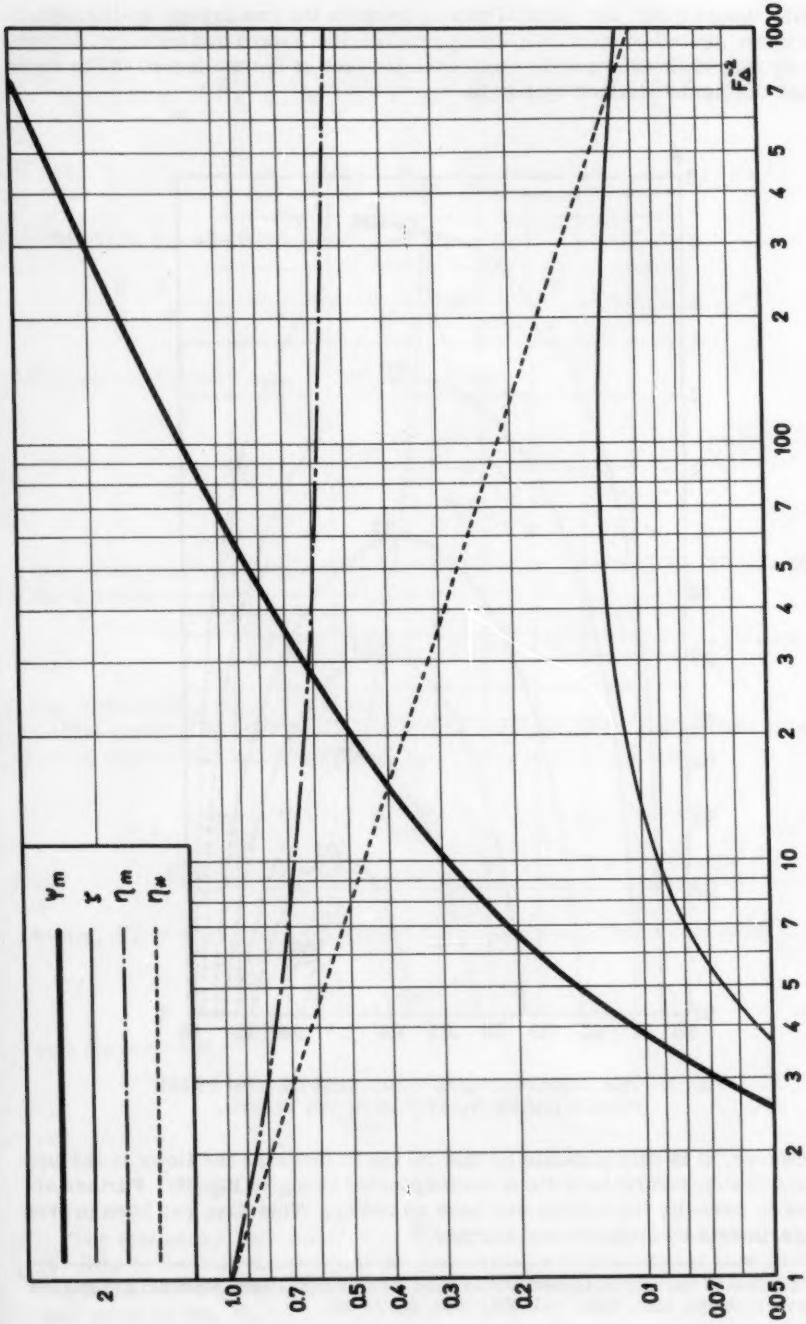

FIG. 3.—ψ_m, η_m, η_* AND ζ VERSUS F_Δ^{-2}

It remains to be determined how this can be utilized in the prototype.

A stream of air bubbles originating from a series of perforated pipes placed on the river bed, at right angles to the direction of the current, will generate an upward flow, that will create the desired mixing if the perforated pipes are covered by salt water. If the flow created by the bubbles exceeds the quantity of salt water that is capable of invading the river mouth in accord with Eq. 26, the salt water will be unable to penetrate far enough to cover the pipes laying furthest upstream. Thus, apart from a slight eddy of brackish water upstream of the tubes, that will be weakened by the natural current in the river, this equipment will form an impenetrable barrier to any salt water stream threatening the river.

THEORY OF UPWARD FLOW CAUSED BY AIR BUBBLES

The upward flow caused by air bubbles has been investigated by Sir Geoffrey Taylor.[4] Assuming that the bubbles are very small, Sir Geoffrey found that air bubbles from a perforated tube on the river bed created a rising, wedge-shaped column of water. The angle of this wedge is about 31°, and the maximum velocity W_o in the vertical current was found to be

$$W_o = 1.9 \sqrt[3]{v\,g} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (27)$$

in which v denotes the discharge of air bubbles per unit length.

Assuming an approximately parabolic velocity profile, and denoting the thickness of the salt water layer h_s, the air bubbles will be capable of mixing per unit length a quantity of salt water

$$q_s = \frac{2}{3}(2\ t\ g\ 15°)\ 5\ h_s\ 1.9 \sqrt[3]{v\,g} = 0.71\ h_s \sqrt[3]{v\,g} \quad \ldots\ldots \quad (28)$$

with the flowing water of the upper layer.

Should this salt water flow, according to Eq. 26, prove insufficient, more tubes can be used. It should be noted, however, that in this case Eq. 28 will hardly give a completely correct result even if the wedge-shaped zones of upward-moving water do not intermix (Fig. 4). For this reason, model tests would be desirable in a detailed design.

NUMERICAL EXAMPLES

The GÖTA ÄLV, a river in the western part of Sweden, has a depth of 6 m and a width of 50 m. The value of Δ is about 0.02.

According to Eq. 26 the vertical flow required to stop the salt water at the barrier is:

$$q_{2,max} = 0.122 \sqrt{(0.02)(9.81)(6)^3} = 0.8 \text{ sq m per sec}$$

---

4 "The Action of a Surface Current Used as a Breakwater," by Sir Geoffrey Taylor, Proc. Roy. Soc. London, Series A, Vol. 231, 1955, pp. 466-478.

If it is estimated that this upward flow must be fully developed $\frac{1}{10}$ h_0 over the river bed and if 8 pipes are used, the air consumption per unit width is found from

$$q_{2,max} = 0.8 = 8\ (0.71)(0.1)(6)\ \sqrt[3]{v\ 9.81}$$

whence

$$v = 0.0013 \text{ sq m per sec per pipe}$$

The total air consumption is

$$V_{total} = 8(0.0013)\ 50 = 0.52 \text{ cu m per sec}$$

It may be mentioned that exceptionally low discharges of 50 and 25 cu m per sec give values for F_{Δ}^{2} of 40 and 160, respectively. The South and South-west

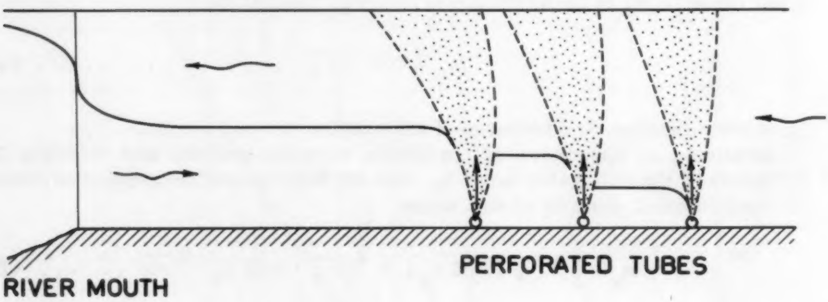

FIG. 4.—PNEUMATIC SALT WATER BARRIER.

passes of the Mississippi River are 11 m deep and the sum of their widths is 750 m. The value of Δ is taken as 0.02. It is found that

$$q_{2,max} = 0.122\ \sqrt{(0.02)(9.81)(11)^3} = 2 \text{ sq m per sec}$$

When 10 pipes are used, the air consumption is found in the same manner:

$$q_{2,max} = 2.0 = 10(0.71)(0.1)(11)\ \sqrt[3]{v\ 9.81}$$

$$v = 0.0017 \text{ sq m per sec per pipe}$$

and

$$V_{total} = 10\ (0.0017)\ 750 = 12{,}7 \text{ cu m per sec}$$

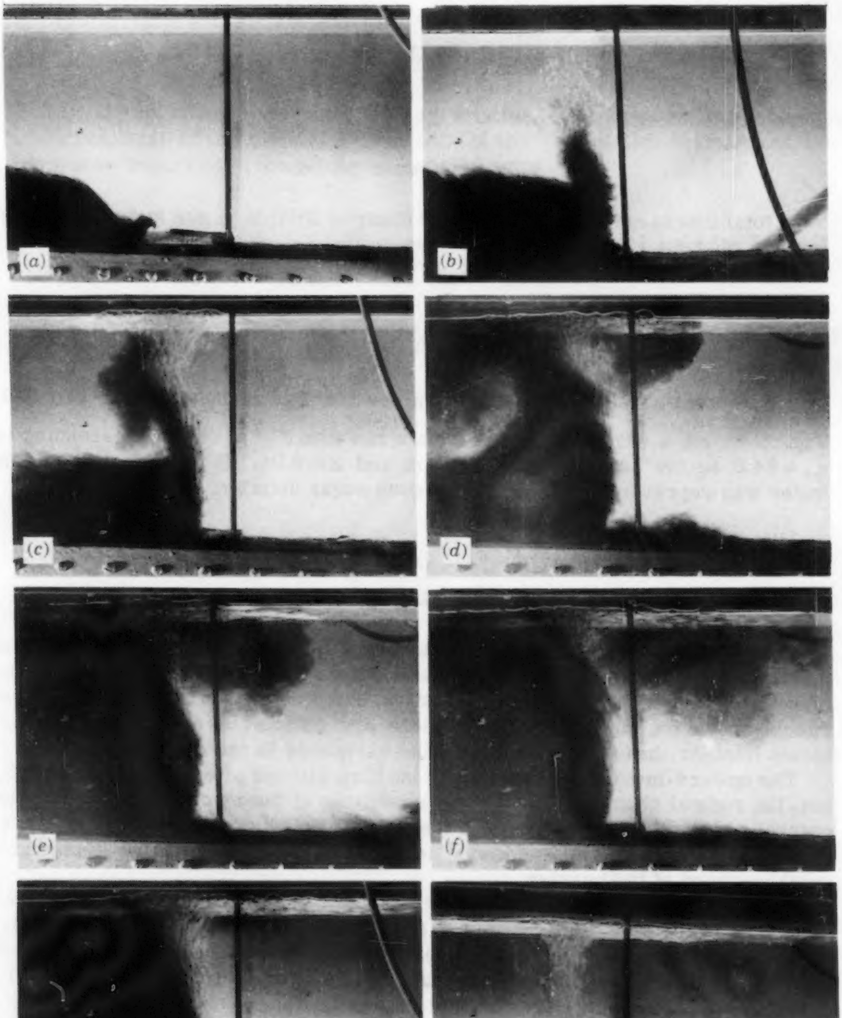

FIG. 5.—MODEL TESTS WITH $F_{\Delta}^{-2} = 125$.

In the Pass A Loutre in the Mississippi River, h_o is 8 m and the width 1000 m. Thus:

$$q_{2,max} = 1.2 \text{ sq m per sec}$$

and corresponding to 10 pipes

$$V_{total} = 10 \text{ cu m per sec}$$

A total discharge in the Mississippi River of 2000 cu m per sec gives values for $F_{\bar{\Delta}}^2$ of about 125 in the three passes.

MODEL TESTS

Model tests have been made with values for $F_{\bar{\Delta}}^2$ of between 100 and 200. Fig. 5 shows a series of photos from a run with $F_{\bar{\Delta}}^{-2} = 125$, corresponding to $q_o = 64.0$ sq cm per sec, $h_o = 23.5$ cm and $\Delta = 0.04$. In these tests, the salt water was represented by means of a dyed sugar solution.

CONCLUSIONS

Pneumatic salt water barriers of the magnitudes found in the preceding numerical examples may be established at costs considerably lower than the construction costs of the usual, less effective, salt water control structures. The running costs of the pneumatic barriers are difficult to estimate in general terms because these costs depend on the variations in the river discharge.

The upward-moving stream of air bubbles will not affect the river traffic, but, the radical change in the current conditions at the river bed will alter the natural sedimentation conditions in the river. Both bed load transport and a considerable part of the suspended load will tend to deposit in the immediate neighbourhood of the barrier.

As the pneumatic salt water barrier is only intended to operate when the discharge in the river is incapable of creating satisfactory fresh water conditions. That is, during periods of greatly diminished material transport, the possible difficulties caused by the barrier with regard to sedimentation will probably be slight.

It was assumed in the introduction that there is an outward surface current. This is not the case at the mouth of well mixed estuaries. Therefore, it is not possible in this way to improve the conditions downstream of the cross-section where the surface current at flood tide becomes zero. However, upstream of this point, the effect will be the same as previously stated. It should be stated that at irregularly shaped river mouthes or at river mouthes where tides play a

dominating role, model tests with the pneumatic salt water barrier will be indispensable.

ACKNOWLEDGMENT

The writer wishes to express his appreciation to Professor, Dr. Fechn. H. Lundgren, head of the Coastal Engrg. Lab. of the Tech. Univ. of Denmark, Under whose supervision the work was carried out.

ADDITIONAL REFERENCE

"Abrupt Change in Width in Width in Two-Layer Open Channel Flow," by Henry Stommel and Harlow G. Farmer, Journ. Mar. Res. Vol. XI, No. 2, 1952, p. 205.

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STUDIES OF A CHANNEL THROUGH PADRE ISLAND, TEXAS

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SYNOPSIS

The problems of tidal hydraulics encountered in design of an artificial inlet from a gulf to an almost tideless bay are numerous. This paper presents the results of several years' observation of tide and current phenomena in such an inlet, and the studies made to determine the best design for a jettied navigation channel through the inlet.

INTRODUCTION

A recent project of considerable interest from the standpoint of tidal hydraulics and beach erosion is the Port Mansfield Channel through Padre Island on the lower coast of Texas. This channel was dredged by the Willacy County Navigation District to provide a navigation route between the coastal bay and the Gulf of Mexico. Two jetties were constructed on the Gulf end to afford protection to the entrance. The construction of this artificial pass afforded opportunity to observe the effects of the disturbance of the natural equilibrium on the Gulf shore by the construction of the jetties and channel. The data and information presented herein were collected by observations extending over a period of approximately 2 yr, following the completion of the Port Mansfield Channel,

Note.—Discussion open until February 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers,,Vol. 86, No. WW 3, September, 1960.

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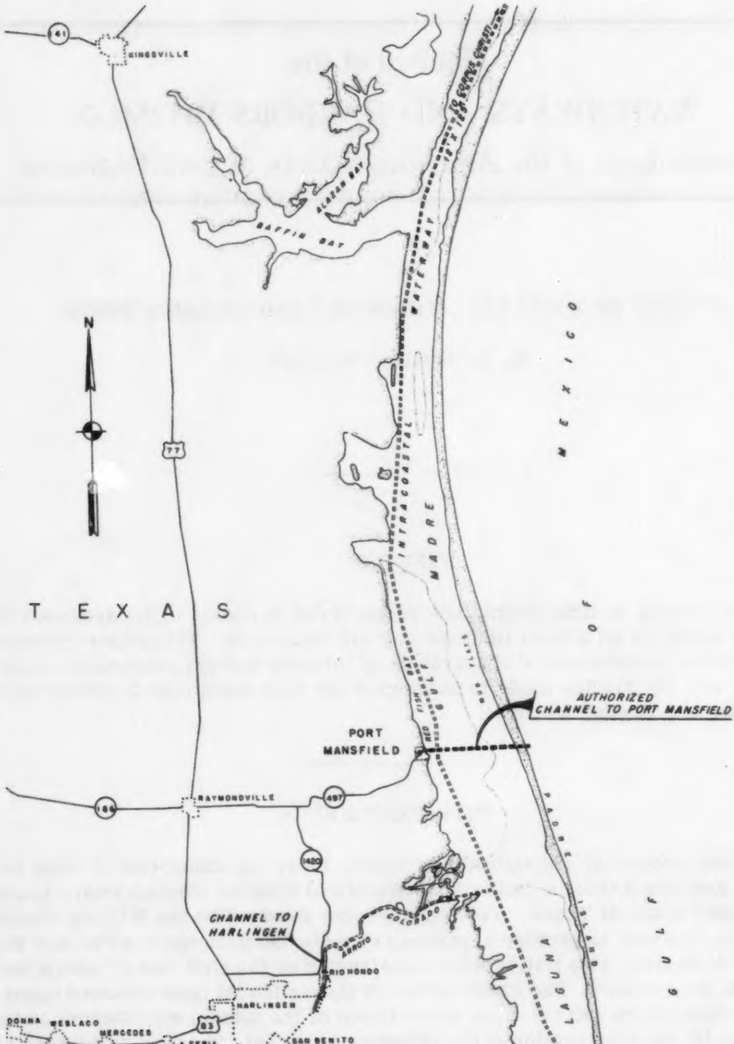

FIG. 1.—PORT MANSFIELD PROJECT LOCATION

and information on studies and conclusions regarding reconstruction of the entrance jetties and channel. The Port Mansfield Channel and vicinity are shown in Fig. 1.

DESCRIPTION

The Port Mansfield Channel lies on the south coast of Texas approximately 93 mi south of Corpus Christi and 38 mi north of Port Isabel. It crosses the Laguna Madre, a long shallow body of water extending along the Texas coast from Corpus Christi on the north to the Brazos Santiago Pass on the south. Laguna Madre, approximately 150 mi long and from 2 to 9 mi wide is separated from the Gulf of Mexico by Padre Island, a virtually uninhabited, low-lying, offshore bar formation. Padre Island varies from 1/2 to 2 mi in width with surface elevations ranging from 2 ft to 16 ft above mean sea level. Higher elevations are all located on a narrow strip of large active sand dunes along the Gulf-beach.

The Laguna Madre is generally divided into two natural bays, separated in the middle by an area of mud flats. The lower bay is approximately 55 mi long and is again divided by a series of shallow flats off the mouth of the Arroyo Colorado which lies approximately 10 mi below Port Mansfield. The bay into which the Port Mansfield Channel enters is locally known as Red Fish Bay and is 28 mi long and 2 to 5 mi wide. Natural depths vary up to a maximum of 8 ft. The ordinary range of tides in the lower Laguna opposite Brazos Santiago Pass is 1-1/2 ft. Tidal effects decrease north of the pass and are barely noticeable in the vicinity of Port Mansfield. Winds cause considerable fluctuations in the water surface elevation. At Brazos Santiago Pass strong north winds may raise the water surface as much as 3 ft above normal tide.

The authorized Federal project for the Gulf Intracoastal Waterway extends through the center of Laguna Madre along its entire length from Corpus Christi Bay to Brazos Santiago. The main channel of the Intracoastal Canal is 12 ft deep and 125 ft wide on the bottom. A branch channel has been extended westward from the main channel in Red Fish Bay to a turning basin at the town of Port Mansfield.

EXISTING IMPROVEMENTS

In September, 1957 the Willacy County Navigation District completed construction of additional improvements to Port Mansfield. The work performed included additional harbor basins, a shallow-draft outlet channel through Laguna Madre and Padre Island to the Gulf of Mexico, and jetties at the Gulf entrance. These improvements are shown in Fig. 2.

The new outlet channel to the Gulf of Mexico extends eastward from the junction of the authorized Federal project channel to Port Mansfield and the main channel of the Gulf Intracoastal Waterway, across Laguna Madre and Padre Island to the Gulf. The channel, from the Federal project to the easterly side of Padre Island, was dredged 10 ft deep and 100 ft wide. From Padre Island through the jetties to the 16-ft depth in the Gulf, the channel was dredged 16 ft deep and 250 ft wide. The new channel from the Gulf Intracoastal Waterway to the Gulf is 8.6 miles in length, and the overall length of channel from the Gulf to Port Mansfield, including the authorized tributary channel and turning basin, is 10.0 miles.

TETRAPOD JETTIES

The two jetties constructed by the local interests were parallel to, and 500 ft distant from the channel centerline and extended from the east shore of Padre

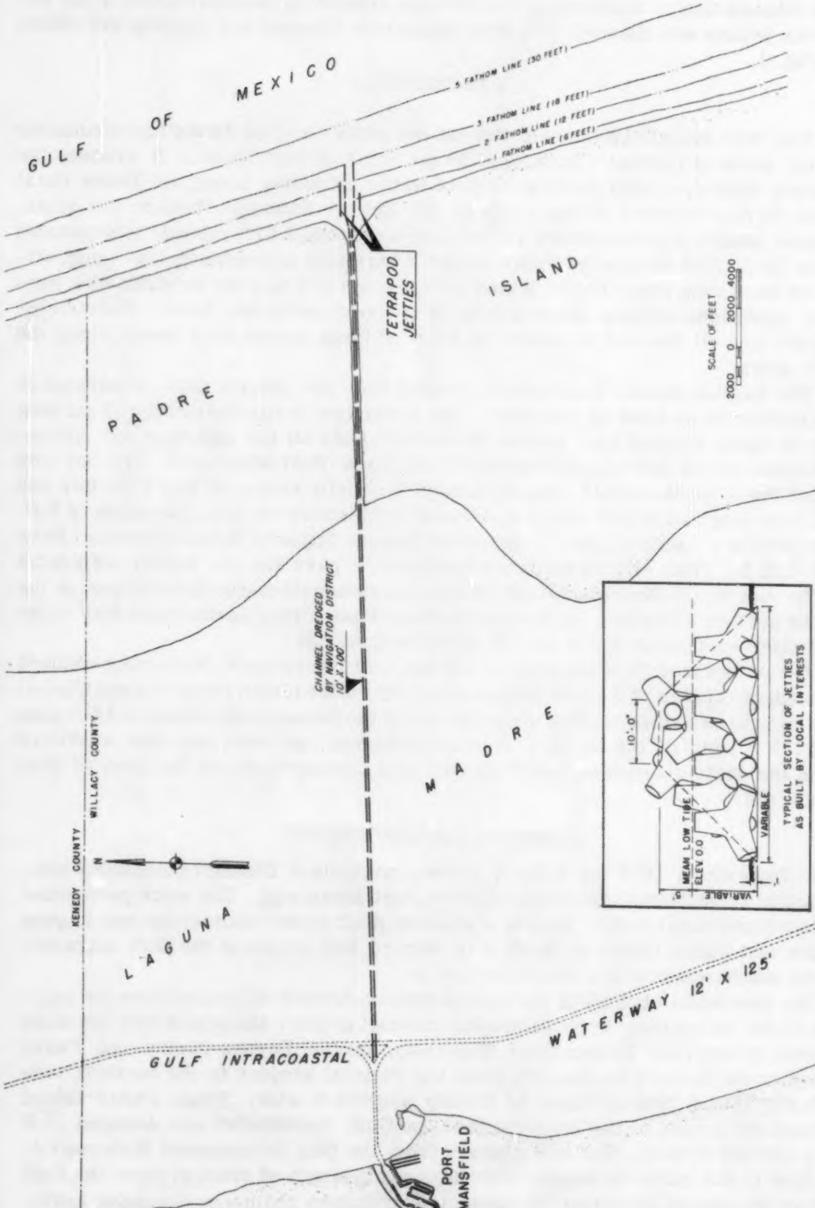

FIG. 2.—LOCAL INTERESTS IMPROVEMENTS, PORT MANSFIELD

Island into the Gulf of Mexico. The north jetty was 1,600 ft long and extended to the 15-ft depth contour. The south jetty was 900 ft long and extended to the 10-ft contour. The crown elevation of both jetties was generally 5 ft above mean low tide and the top width was 10 ft. The jetties were formed by placing precast concrete pieces of geometric design. These pieces, known as tetrapods, were placed on the sand, without a stone blanket to distribute the load or prevent erosion. The tetrapods consist of four symmetrically placed truncated conical legs emanating from a common center. The unit weights of the blocks used in the Port Mansfield jetties were 5, 8, and 16 tons. Side slopes of the jetty mounds were approximately 1 vertical on 1 horizontal. A typical section of the jetty is shown in Fig. 2.

Subsequent to completion of the locally constructed improvements, extensive deterioration of both jetties occurred. In November, 1957 severe storms produced a period of very heavy seas and severe settlement of the jetties. Scouring currents washed away material at the inner ends of the jetties and resulted in shore line recession sufficient to flank both jetties. With the effectiveness of the jetties destroyed, extensive shoaling occurred in the channel between the jetties, which had been dredged to a depth of 16 ft. Field surveys were made during February, 1958 to determine the present condition of the improvements constructed by the local interests. No material changes were found in any of of the facilities except in the vicinity of the Gulf entrance, where the jetties and entrance channel were found to be badly deteriorated and not functioning so as to be useful to navigation.

No evidence was found of scattering or displacement of the tetrapods, but very extensive subsidence had occurred. Indications are that the subsidence of the jetties resulted from a combination of scour and compression of the base materials. Channels from 2 ft to 4 ft below the Gulf bottom had been scoured along the outer two-thirds of both sides of the north jetty and the north side of the south jetty. Available borings and probings that were made during the survey indicated that the base materials for the jetties consist of a sand layer, ranging from a few inches to several feet in thickness, overlying a soft plastic clay. Along the outer half of the north jetty, where subsidence had been most severe, no sand layer was found.

Subsidence affected the north jetty considerably more than the south jetty. Except for short reaches, the entire north jetty is now below the normal water surface with the settlement progressively greater toward the outer end, where the highest points on the tetrapods are from 3-1/2 ft to 7 ft below the water surface. From the midpoint of the north jetty, inshore for a distance of approximately 350 ft, a number of tetrapods remain above the water. In this reach a few inches of coarse sand were found overlying the soft clay. Considerably more sand was found along the base of the south jetty than along the north jetty. Although the entire south jetty has subsided, many of the tetrapods are still visible above the water over most of its length. Photographs of the visible portions of the jetties in February, 1958, seven months after completion, are shown in Fig. 3. Recent inspections indicate little recent subsidence and the jetties may have become stabilized, or they are subsiding at a very slow rate.

The high void ratio in the tetrapod jetties, which has been estimated at 50%, results in very low efficiency in intercepting and holding littoral drift materials and in confining tidal currents. When the jetties were outflanked by shore line erosion, their usefulness for any function, except as a breakwater, was destroyed, even though subsidence had not occurred. In the deteriorated condition, the jetties afford no protection to the dredged channel from a maintenance standpoint. This is evident from the rapid shoaling that completely obliterated the

dredged channel seaward from about the original Gulf shore line. Moreover, without jetty protection, natural shore processes probably will tend to close this channel in the absence of major scour forces from storms or hurricanes.

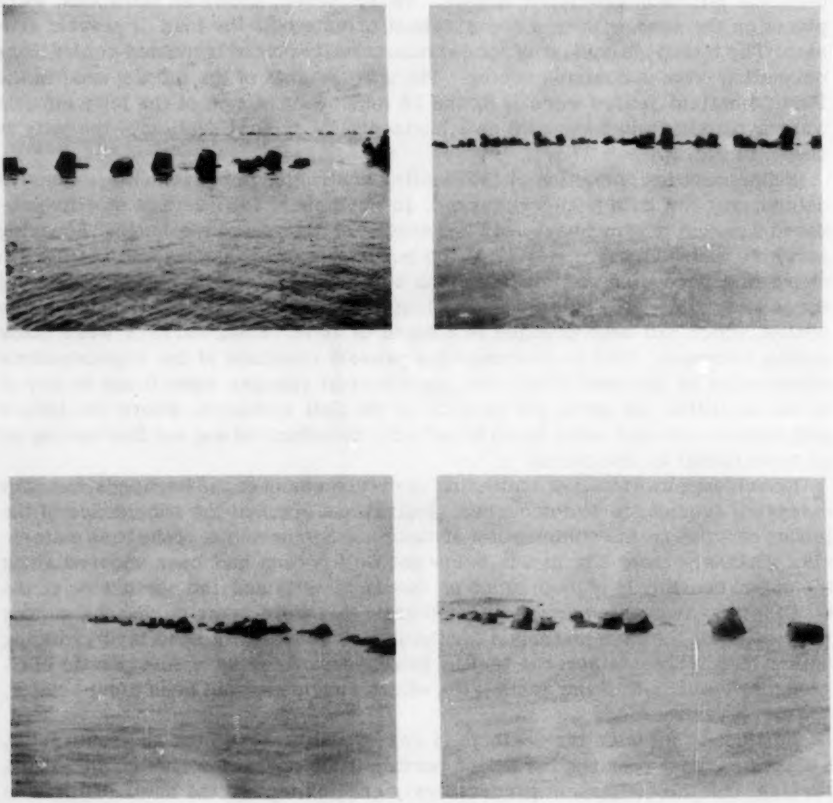

FIG. 3.—CONCRETE TETRAPOD JETTIES AT GULF ENTRANCE TO PORT MANSFIELD, TEXAS

A series of aerial photographs of the jetties and shore line near the entrance are shown in Fig. 4.

SHORE LINE CHANGES

The Gulf shore line north of the entrance and the channel banks for several thousand feet inland have undergone extensive erosion and accretion cycles since completion of the channel and jetties. The series of photographs in Fig. 4 and

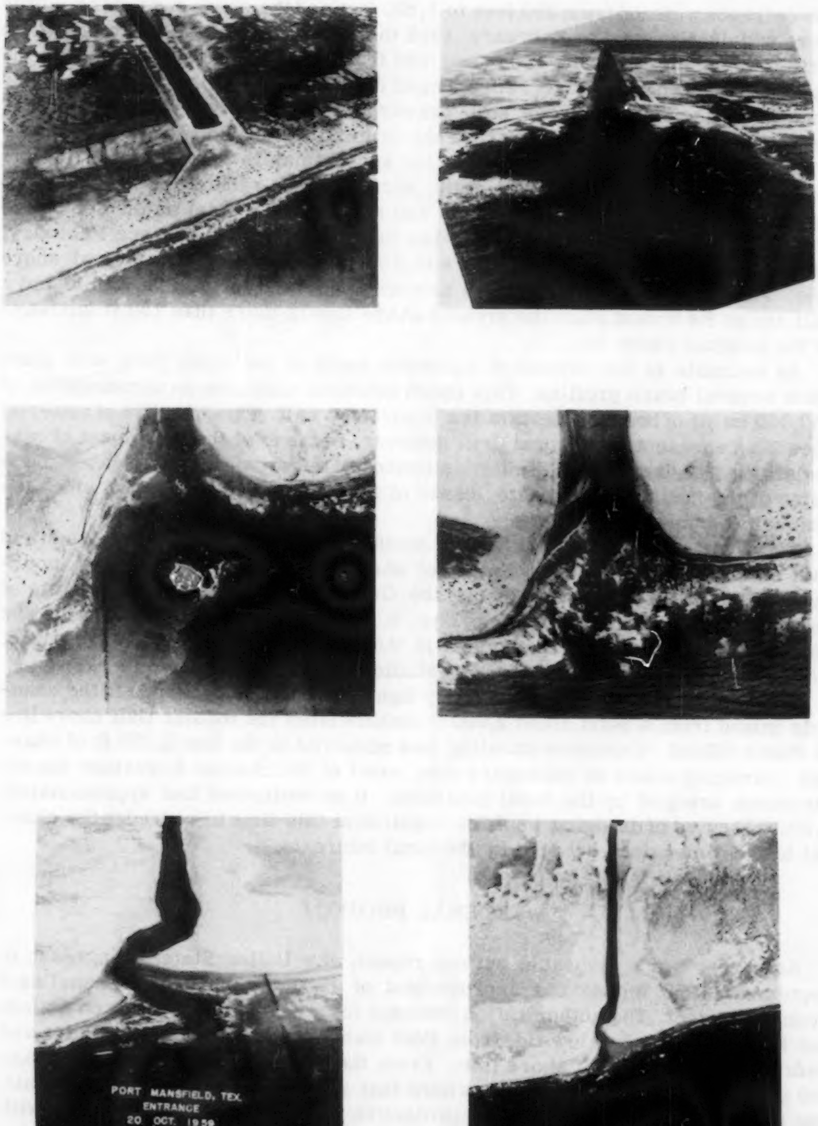

FIG. 4.—CONCRETE TETRAPOD JETTIES AND CHANNEL THROUGH PADRE ISLAND PORT MANSFIELD, TEXAS

the comparative shore lines in Fig. 5 show these changes. Initially the width of the entrance widened from 250 feet to 1,800 feet and the shore ends of the jetties were widely flanked. In February, 1958 the shore ends of the north and south jetties were, respectively, 100 ft and 250 ft seaward of the shore line. In the spring of 1958, however, accretion began on both sides of the channel. At the south jetty the shore line has built seaward 700 ft from its farthest landward location and now is 250 ft seaward of the original shore line. The accretion extends southward along the Gulf shore for approximately 3,000 ft.

At the inner end of the north jetty, accretion has rebuilt the shore line to connect with the inner end of the jetty and has formed a point on the south side of the jetty. North of the jetty there has been some accretion since February, 1958 but the present shore line is still 100 ft landward of the original shore line. The erosion of the Gulf shore extends northward for almost a mile and a half and at its widest point the present shore line is more than 150 ft landward of the original shore line.

An estimate of the volume of accretion south of the south jetty was made from several beach profiles. This rough estimate indicates an accumulation of 250,000 cu yd of beach materials in a year and a half. This quantity of material does not represent the littoral drift however, because of the movement of materials through the jetties. Similar estimates of the erosion that has taken place north of the north jetty indicate losses of the order of 400,000 cu yd since the channel was constructed.

Fig. 6 is a shore line and offshore depth change study of the Port Mansfield Gulf entrance channel area. This plot shows the historical landward movement of the mean high water line on the Gulf side of Padre Island, the rate of retreat being approximately 9 ft per yr. It is possible that land subsidence and beach depletion are significant factors in the indicated losses from Padre Island. However, such factors have not been studied in detail.

Surveys indicate that comparatively light shoaling has occurred in the channels inland from a point about 2,000 ft inshore from the former Gulf shore line of Padre Island. Extensive shoaling has occurred in the last 2,000 ft of channel. Dredging would be necessary over most of the channel to restore the dimensions dredged by the local interests. It is estimated that approximately 1,000,000 cu yd of dredging would be required at this time to redredge the channel to the dimensions dredged by the local interests.

FEDERAL PROJECT

As a result of a favorable survey report, the United States Congress, in September 1959, authorized improvement of the Port Mansfield channel as a Federal project. The authorization provides for a channel 100 ft wide on bottom and 14 ft deep at mean low tide from Port Mansfield across Laguna Madre and Padre Island to the Gulf shore line. From the shore line the channel is made 250 ft wide and 16 ft deep, out to where that natural depth occurs in the Gulf. The project also provides for dual protective jetties at the Gulf entrance, with the north jetty to be 2,300 ft long and the south jetty to be about 2,270 ft long. Planning studies to determine the most feasible type of jetties to construct were

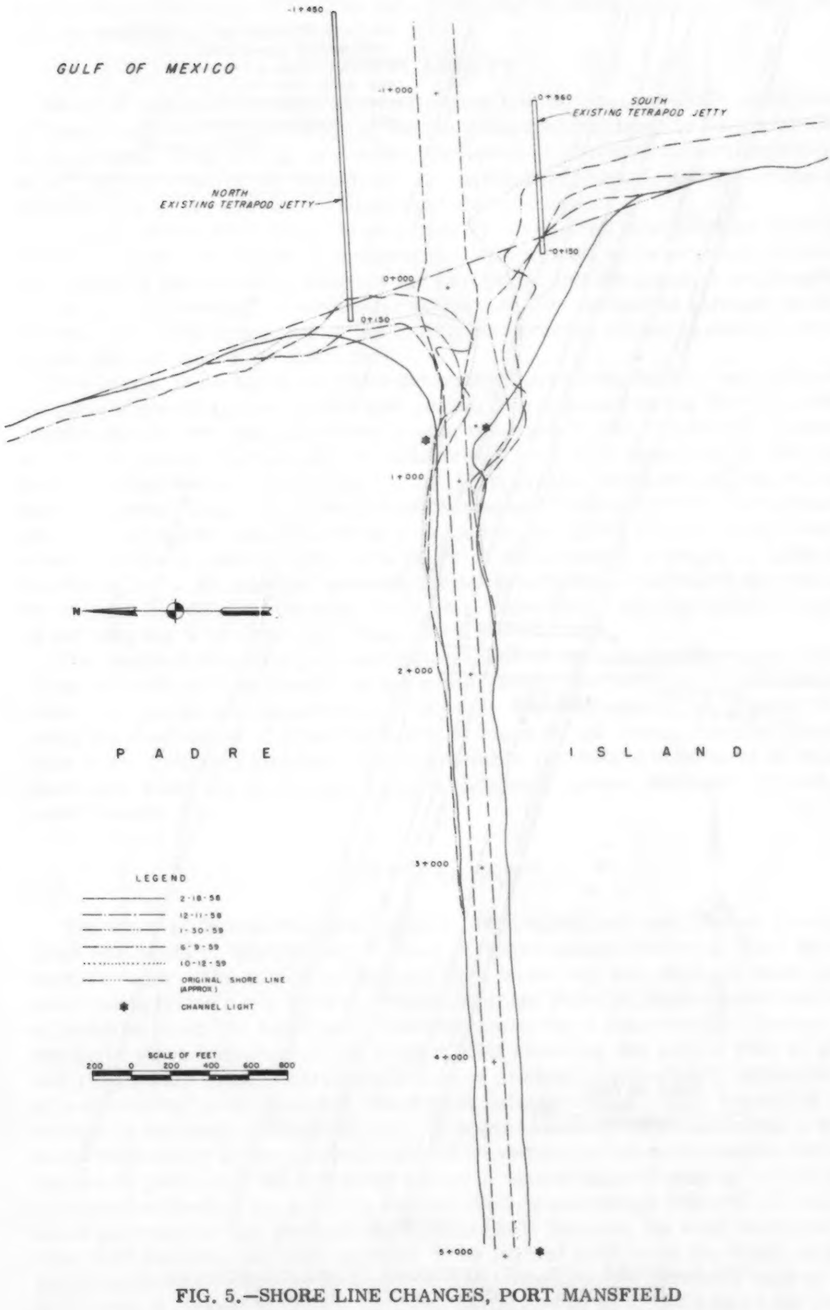

FIG. 5.—SHORE LINE CHANGES, PORT MANSFIELD

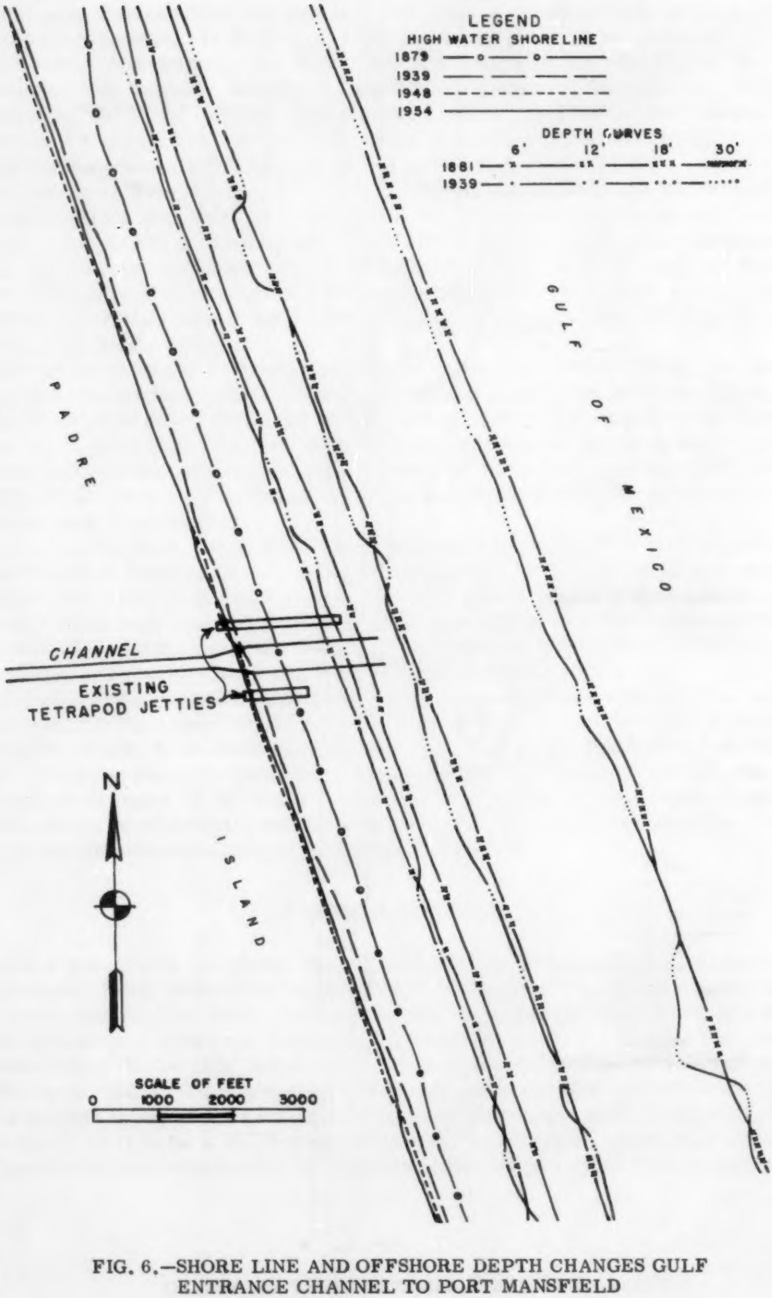

FIG. 6.—SHORE LINE AND OFFSHORE DEPTH CHANGES GULF ENTRANCE CHANNEL TO PORT MANSFIELD

completed on March 1, 1960. The data presented in the remainder of the paper are the results of the design studies.

JETTY LENGTH

Study of the most feasible types of jetties for the shallow-draft navigation entrance included consideration of the two principal functions to be performed by the jetties. These are (1) preventing the flow of littoral drift materials through or around the ends of the jetties into the navigation channel, and (2) acting as breakwaters to permit safe navigation of the entrance.

A relatively impervious and sand tight jetty is required to prevent the passage of littoral drift materials. Spacing of the jetties must be determined by consideration of the optimum distance for navigation and the location and lengths should provide desirable breakwater action and safe navigation through the entrance. The 1,000-ft spacing of the locally constructed jetties is considered to be satisfactory for these functions.

The length of the north jetty was determined primarily by the requirements considered necessary for breakwater action. The seaward end of the north jetty constructed by the local interests is approximately 1,300 ft from the original shore line, which was considered satisfactory from this standpoint. It was determined that the new north jetty for the 16-ft project entrance channel should have an overall length of 2,300 ft. Studies also indicated that for an 18-ft project channel, the north jetty should be extended to the 18-ft natural depth, which would require a seaward extension of 300 ft and an overall length of 2,600 ft. Similarly, for a 36-ft project channel, the north jetty should extend to the natural 30-ft depth in order to act effectively as a breakwater, and the overall length of the jetty for a 36-ft project should be 5,240 ft.

The length of the south jetty was determined by the anticipated accumulation of the littoral drift materials on the south side of the jetty and by the breakwater and navigation requirements. A study and analysis of the littoral drift along the Gulf shore of Padre Island was made by the Beach Erosion Board. This study indicated that there is considerable northward movement of beach materials along the shore that would accumulate against the south side of an impermeable jetty.

LITTORAL DRIFT

The study of beach characteristics at Port Mansfield and Brazos Santiago Pass was made to determine the effect of impermeable jetties at Port Mansfield entrance. The comparative mean high water line and offshore depth contours north of the north jetty at Brazos Santiago Pass indicate substantial loss of material from the beach and foreshore zones for a considerable distance to the north since initiation of the project (data covering the period 1881 to 1939 and 1948). This evidence strongly indicates predominant northerly littoral drift at a substantial rate. However, these data indicate only a minor amount of accretion to the south of the south jetty. It appears to be quite probable that a very large percentage of the littoral material movement to the north passes through the south jetty into the entrance channel. Maintenance dredging records of dredging for the Gulf bar and jetty channel indicate an average removal of 352,000 cu yd per year for the period from 1936 to 1957. Because the most likely major source of shoaling material appears to be littoral drift from the south, and as the erosion rate to the north supports this reasoning, net northerly rate of littoral drift at Brazos Santiago Pass was estimated to be 300,000 cu yd per year.

The general shore line at the Port Mansfield entrance channel has virtually the same orientation as at Brazos Santiago with comparable wave energy distribution. For design purposes the estimated 300,000 cu yd annual drift rate at Brazos Santiago Pass was considered to be applicable to the Port Mansfield site. Based on this estimate of littoral drift it was estimated that the accumulation would move the mean high water line seaward along the jetty over 10-yr periods as shown in Table 1 and in Fig. 7.

A study of the characteristics of the beach materials indicated that the outer end of the south jetty should be positioned to provide a 1-on-60 bottom slope from the mean high water line to the depth to be maintained in the jetty channel. With mean high water at elevation 2 ft above mean low tide and the recommended jetty channel depth of 16 ft below mean low tide, the outer end of the jetty should be 1,080 ft seaward of the mean high water line. This criterion would require a total jetty length of 2,270 ft to retain the littoral drift for a period of 10 yr. It is considered that for the 16-ft jetty channel, the south jetty should initially be built to contain a 10-yr accretion of littoral materials. This would afford a 10-yr period in which to study the actual rates of accretion of littoral materials and the effectiveness of the jetty in retaining the materials. Further, it is believed possible that development of the shore north of the north jetty over the next 10 yr may increase the land values so that bypassing the beach materials around the jetties may be warranted to prevent the erosion that would result from loss of littoral drift caused by the jetties. The south jetty would have an overall length of 2,270 ft. To provide satisfactory breakwater action for shallow-draft navigation, a jetty crown height of 5 ft above mean low tide is considered necessary.

TABLE 1.—LOCATION OF MEAN HIGH WATER LINE

Period after construction of the south jetty, in years	Length along south jetty, in feet
10	600
20	1,000
30	1,300
40	1,500
50	1,740

The length of the south jetty for deeper channels was also determined from consideration of the same factors. At an assumed angle of repose slope of 1-on-60 for the littoral materials, a depth equivalent to the entrance channel depth of 20 ft for an 18-ft project would be reached at a point 1,320 ft seaward from the high water line of the 10-yr sand accumulation south of the jetty. A jetty to that point would be 2,510 ft long. However, since the existing natural bottom-slope averages somewhat flatter than 1-on-60, a jetty length of about 2,930 ft would be required to reach the natural 18-ft depth contour. A length of 2,490 ft was considered to be the optimum length of the south jetty for a 20-ft entrance channel. This jetty would terminate in a natural depth of about 16-1/2 ft of water. Similarly, a 36-ft project with an entrance channel 38 ft deep would require a south jetty length of 3,590 ft to reach an equivalent depth of 38 ft on the assumed slope of 1-on-60. Because of the flatter natural bottom-slope, a total length of 5,130 ft would be required to reach the natural 30-ft depth contour. The latter length would be more desirable for furnishing breakwater protection for deep-draft

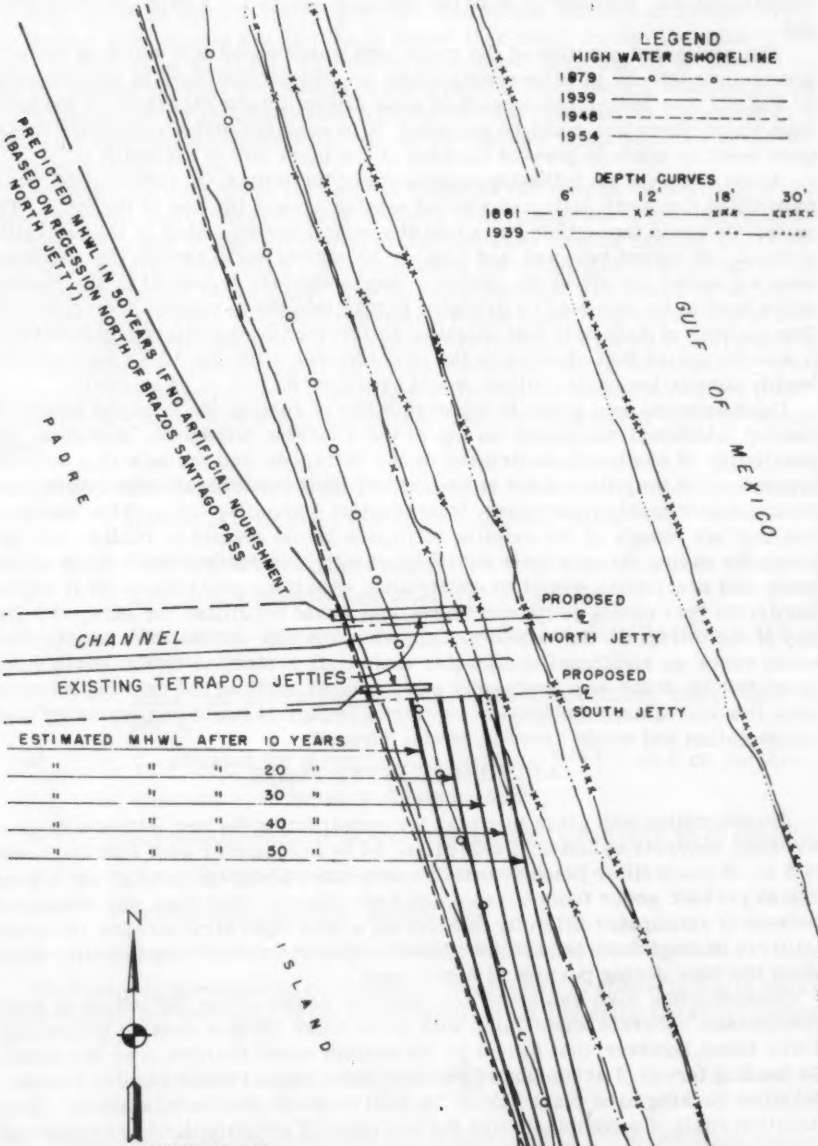

FIG. 7.—PREDICTED SHORE LINE AND OFFSHORE DEPTH CHANGES

vessels and was considered to be the optimum length for a 38-ft entrance channel.

The estimated position of the mean high water shore line north of the proposed north jetty 50 yr after construction of entrance structure is also indicated in Fig. 7. For lesser increments of time proportionate recession of the mean high water shore line could be expected. It is considered that protective measures must be taken to prevent flanking of the inner end of the north jetty.

At the end of 10 yr, following completion of the jetties, the littoral drift would have filled the south jetty and started moving around the end of the jetty. The materials would then either be moved shoreward and deposited in the navigation channel, be moved seaward and lost, or be moved north across the entrance, where it would not affect the project. Any materials deposited in the channel would have to be removed by dredging to maintain the navigable project depths. The quantity of materials that might be so involved is extremely problematical. It was estimated that shoaling in the jetty channel, after the 10-yr interval following completion of the jetties, would average 100,000 cu yd annually.

Consideration was given to the possibility of raising the tetrapod jetties by placing additional materials on top of the concrete tetrapods. However, the possibility of continued settlement of the tetrapods and the lack of a suitable foundation for the jetties made an estimate of their condition at some future construction date highly speculative. Conservative estimating dictated the assumption that not enough of the existing tetrapods would remain to realize any appreciable saving through their use. Accordingly, the designs were made on the basis that new jetties would be constructed with their centerlines 50 ft southward from the existing jetty centerlines, and would not utilize the tetrapod jetty. Any of the tetrapods that would remain when the new jetties were constructed would serve as additional breakwater and scour protection on the north sides of the jetties. If the new south jetty were located north of the existing tetrapod jetty, it would be expected that any remaining tetrapods would be covered by sand accumulation and would serve no useful purpose.

ALTERNATE JETTY PLANS

Consideration was given to a plan for constructing the new jetties with prestressed concrete cylindrical cell piles, 54 in in diameter with side thickness of 5 in. A stone filter blanket would be necessary along the jetty at the ground line to prevent scour from current and wave action. This plan was discarded because of anticipated difficulty in securing a sand tight structure and the probability of damage from impact and abrasion by movement of the protective stone along the base during periods of heavy seas.

Consideration also was given to a plan for constructing the jetties of solid prestressed concrete sheet piles with stone filter blanket erosion protection. It was found, however, that except in the shallow water reaches near the shore, the loading forces of accumulated sand and wave impact would require substantial stone backing to be placed along the wall to insure structural stability. Construction costs of a combination of the two types of construction were estimated to be higher than for a stone jetty without the concrete sheet piles. If suitable construction equipment were readily available, it is possible that slight savings might be effected by use of the concrete sheet piles onshore and in the shallow water reaches offshore.

JETTY DESIGN

The proposed design of the jetties differs from the design of existing jetties along the Texas coast in several aspects. One of these aspects is the positive

features aimed at making the jetty impervious to the passage of littoral drift. The 3-on-1 side slopes are generally flatter than usual so that the size of the cover stone can be smaller. The cover stones are to be placed at random so that the face of the jetty will be rough and irregular. The design will permit the use of limestone as an alternate to granite for the cover stone. A cretaceous limestone of suitable quality and size can be obtained from quarries in the vicinity of Austin, Texas.

The structural design of the jetty is based on the results of analysis of the foundation soil, and the pressures on the jetty from design waves and tide heights.

Storm Tide and Waves.—The storm tide and wave heights used to design the jetty were determined from a consideration of the magnitude of hurricanes that can affect the lower Texas coast. The storm-tide height was based on the water level response at Port Mansfield that would result from a standard project hurricane crossing the coast on a path normal thereto and with its center south of Port Mansfield a distance equal to the radius of the region of maximum wind speed. The characteristics of the standard-project hurricane have been determined by the United States Weather Bureau, Dept. of Commerce (USWB) as follows:

CPI = Central pressure, 27.50 in.;

R = Radius to maximum winds, 13.5 nautical miles;

T = Forward speed of storm, 11 knots;

V_{cx} = Maximum cyclostrophic wind, 112 mi per hr;

V_{gx} = Maximum gradient wind (V_{cx} adjusted for coriolis affect), 110 mi per hr;

V_x = Maximum wind speed 30 ft above the water ($0.865\ V_{gx}$), 95.1 mi per hr;

W_m = V_x adjusted for direction and including $1/2\ T = 93.8$ mi per hr;

N_m = Wind set up of water at 6 fathom depth;

$V/\bar{c}$ = Shoreward velocity of storm divided by mean free wave speed;

F/L = Fetch divided by width of continental shelf;

S = Water level response factor, and

K = A constant based on known storm tide relationship, 1.62.

The formula, developed by the Beach Erosion Board in connection with hurricane studies, for determining the water level set up and using the above factors is:

$$N_m = K\,W_m^2\,S \qquad (1)$$

The values used in this formula give storm tide level of 6 ft above mean low tide in the Gulf at Port Mansfield. The frequency of occurrence of the standard project storm is estimated at once in 100 yr.

A study of the possible wave heights that could be generated at the outer end of the jetties by the design storm indicate that the depth of water would control the wave height, because the maximum wave that would break in the available water depths could be generated by the design storm. The maximum wave, that is the average of the highest 10% of the waves in the wave spectrum, is estimated at 12 ft for the standard project storm at the outer end of the jetty. The wave would be reduced to 4 ft at the shore line because of the shallow water.

Foundation Investigations.—In December, 1959 and January, 1960 fourteen undisturbed borings were made along the proposed new jetty alignments. Seven borings were drilled on land and seven in the water. The logs of the borings are shown in Fig. 8.

The shore end of the jetties will rest on approximately 5 ft to 7 ft of fine sand. The sand strata decreases in thickness seaward and is absent from the foundation areas of the outer ends of the jetties. Occasionally sand is deposited by the shore currents over the soft clay beneath the outer ends of the jetties. Below the sand strata on the shore end of the jetties is a medium stiff clay lense 6 ft to 8 ft thick with a soft clay strata 10 ft to 12 ft thick below it that extends the entire length of the jetties. Below the soft clay strata a 3 ft to 4 ft thick soft clayey to silty sand strata is encountered. Below the soft clayey or silty sand strata, hard clays and dense sands are encountered to a depth of at least 50 ft below sea level.

Moisture content, liquid limit, and grain size distribution tests were made on all samples. Density tests were also made on all undisturbed samples. Consolidated-drained and unconsolidated-undrained shear tests were made on typical samples from the various foundation strata. Unconsolidated-undrained triaxial shear tests were performed on typical foundation materials to check the test results obtained from the direct shear tests. Consolidation tests were also performed on the typical foundation materials. Composite consolidation curves for the typical foundation materials were drawn using the average of several curves made on each material.

The soft clay strata is about 12 ft thick beneath the outer ends of the jetties. The average moisture content is about 50% and the unit dry weight is about 72 pct. Both consolidated-drained, and unconsolidated-undrained direct shear tests were made on the soft clay strata. Triaxial unconsolidated-undrained shear test results were used to check the direct shear test results for final design. Values used for design were selected from these results.

Jetty Section.—Studies based on the data obtained from subsurface exploration and testing indicate that a rubble mound jetty can be constructed at the Gulf entrance to the Channel to Port Mansfield without excessive settlement. The section has been designed to meet the following major requirements: (a) Prevent flow of littoral drift material through the jetty; (b) Be stable against a design wave height of 12 ft; (c) Be safe from shear failure in the soft foundation material; and (d) Reduce wave action between the jetties to a minimum.

The jetty section that most adequately meets the above requirements would have a relatively impervious core with successive layers of larger stones to prevent piping. The side slopes must be flat enough to prevent foundation failure. A jetty section consisting of the featuring elevation of the top of jetty of + 8.0 ft MLT, top width of 16 ft, and side slopes of 1-vertical-to-3-horizontal is proposed. A stone blanket 3 ft thick will be provided under the entire jetty and extending from 2 ft to 15 ft beyond the toe. This blanket will be composed of crushed stone 1/2 in. to 200 lb in size. Typical sections of the jetty are shown in Fig. 9.

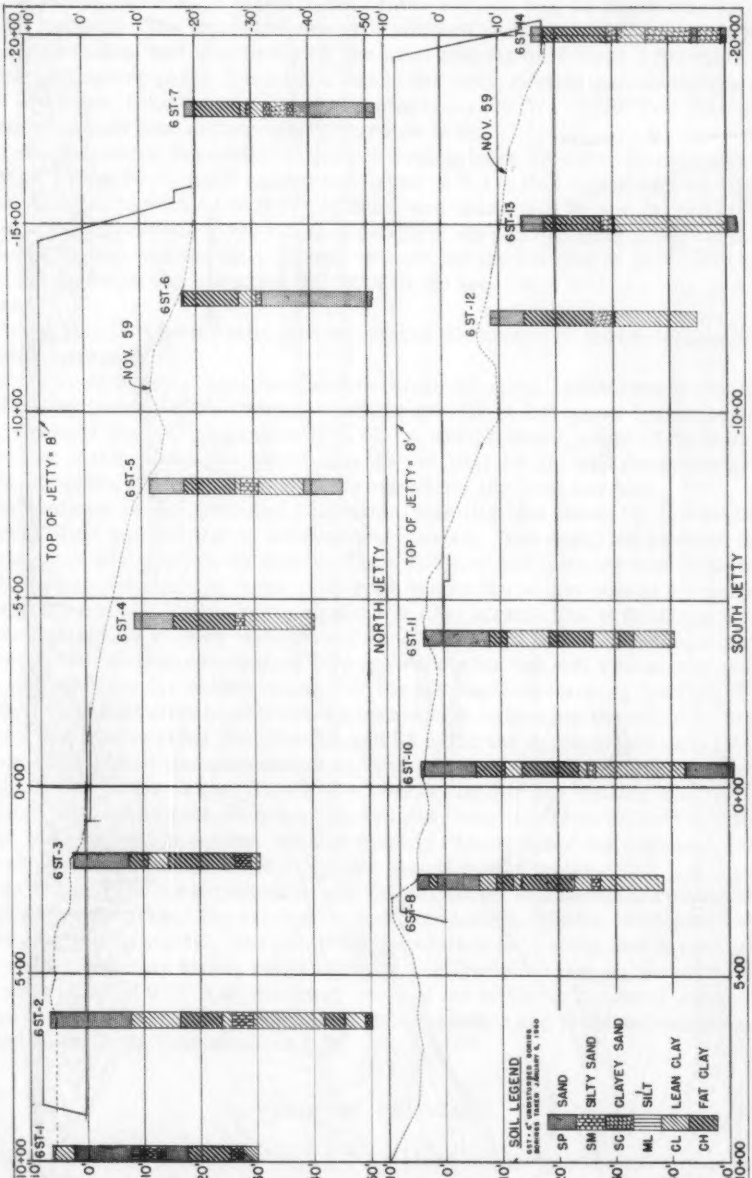

FIG. 8.—BORINGS AND JETTY PROFILES

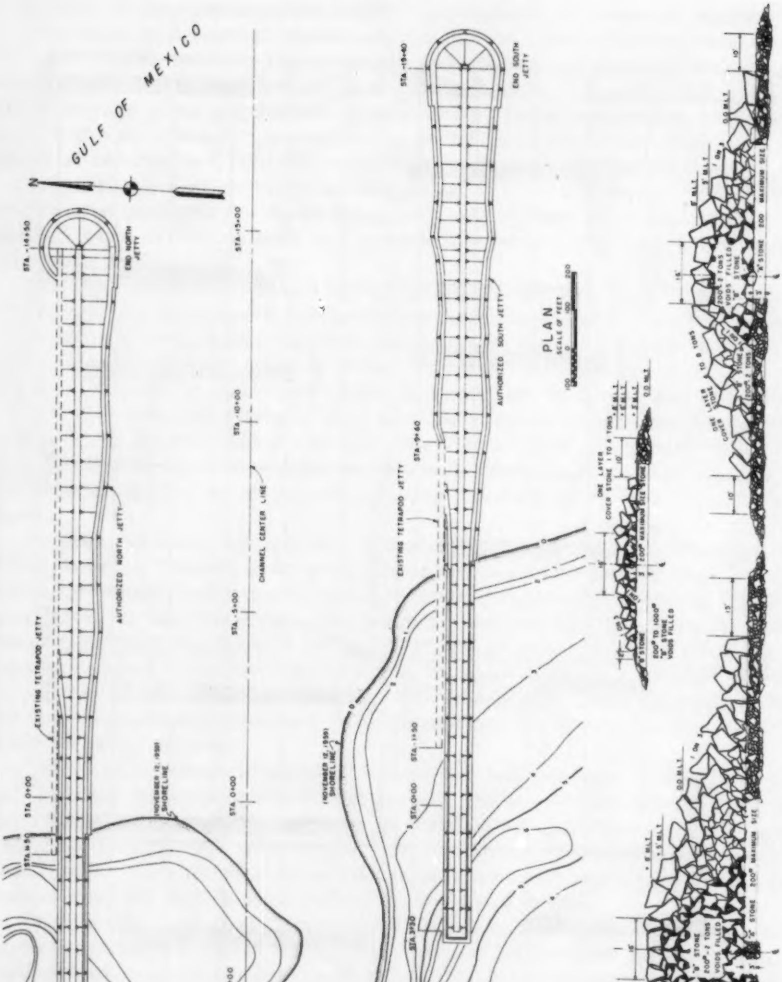

FIG. 9.—TYPICAL JETTY CROSS SECTION AND PLAN

The core will consist of reasonably graded stones from 200 lb to 4,000 lb in weight. In order to provide a relatively impervious core that will prevent flow of sand through the jetty, voids in the center section will be filled with 1/2 in. to 4 in. material. The top of the core will be constructed to + 5.0 ft MLT to provide an elevation that will be above the area subject to littoral drift after expected settlement of the foundation and in the jetty section due to rearrangement of stones. Smaller core stones ranging from 200 lb to 1,000 lb will be used on the minimum size section constructed on land.

Cover stones for the section of jetty extending from the shore line at elevation 0.0 MLT to the 10-ft depth will be one layer of 2-ton to 6-ton stones and, from the 10-ft depth to within 100 ft of the outer end there will be one layer of 4-ton to 8-ton stones. On the outer 100 ft of jetty and the end section 2 layers of 6-ton to 10-ton stones will be used. Cover stones for the section of jetty extending from the shore line at elevation 0.0 MLT to the inner end will be 1-ton to 4-ton stones.

The centerline of each jetty will be located 50 ft south of the centerline of the existing tetrapods.

The size of cover stones has been determined using Irribarrens formula as modified by Hudson. The average specific gravity of limestone investigated is 2.53 whereas that of the granite is 2.63. A design wave height of 12 ft at the outer end of the jetties has been used. As the wave height will decrease as the shore is approached, smaller stone are used for the land sections.

An analysis of the jetty and foundation stability was made by the modified Swedish Slide method and by Jurgenson's method. The modified Swedish Slide method gave the least safety factor. The section of the jetty deemed to be most nearly critical was at the outer jetty ends where the height was at a maximum and the base of the jetties rests on the soft clay strata. The soft clay strata is approximately 12 ft thick with strong sand and clays below. Unconsolidated-undrained direct shear tests give average values for the soft clay strata of 0.25 tons per sq ft for the cohesion and 5.2° for the angle of internal friction. Consolidated-drained direct shear tests give average values for the soft clay strata of 0.17 tons per sq ft for the cohesion and 23.3° for the angle of internal friction. The unconsolidated-undrained triaxial shear tests on this material give average values of 0.275 tons per sq ft for the cohesion and 0.0° for the angle of internal friction. The safety factors using test results from the three different types of shear tests are all adequate. Analysis using results from the unconsolidated-undrained triaxial shear tests gives the lowest safety factor.

The majority of the expected foundation settlement will be due to consolidation of the soft clay strata. The settlement has been computed using the consolidation curves for this material. The computed foundation settlement due to consolidation of this soft clay strata ranges from a negligible amount on the shore ends to a maximum of 0.75 ft at the outer ends of the jetties. The total settlement of the jetty due to the consolidation of the foundation and jetty materials is expected to be of the magnitude of 3 ft.

CHANNEL SHOALING

Surveys of the channel in the 2 1/2 yr since its completion have been made on eight occasions. These surveys varied from centerline profiles by fathometer to cross sections with sounding lead. A study of the results indicates that following the initial erosion at the Gulf entrance when there apparently was erosion in a large part of the channel, shoaling has occurred throughout the channel. The

average shoaling increased from about 4 cu yd per ft in the 14,000-ft reach east of the Intracoastal Waterway, to 8 cu yd per ft across the shallow reaches of Laguna Madre, and about 16 cu yd per ft in the 6,000-ft reach west of the Gulf shore line. The estimated shoaling in the channel between the Intracoastal Waterway and the Gulf shore amounted to about 300,000 cu yd annually.

TIDE OBSERVATIONS

Recording tide gages have been operated at four locations on the channel since February 1958. The gages were located at the Gulf entrance, the Gulf Intracoastal Waterway and two intervening locations on Padre Island. Difficulties with the gage mechanism prevented continuous recordings and, because the rapid shoaling on the bar affected the tidal flow into the channel, firm conclusions on tidal flows in the channel cannot yet (1960) be made.

It has been established, however, that the water level in the bay is generally 1 ft to 2 ft higher than in the Gulf, except during short periods of the normal high tide cycles in the Gulf and during abnormal high tides induced by storm periods in the Gulf. The normal range of tide cycles in the bay is insignificant, being generally on the order of 0.1 ft to 0.3 ft. Sustained periods of south winds cause a slow, prolonged rise in the water level of the bay, which is equally slow to recede after the winds have subsided. During these periods the water level may be raised from 1 ft to 2 ft above normal during a period of two weeks or longer.

The observed currents in the small, meandering channel, which is being maintained at the Gulf entrance by natural forces, have been outbound practically the entire time. Velocities through the constructed reach of the channel range from slight to moderate, depending mostly upon the water level in the Gulf. On rare occasions, high tides in the Gulf will result in a reversal of the current for short periods of time. Farther inland, where the channel section has not been reduced appreciably by shoaling, current velocities are barely noticeable. Based upon the incomplete data, it appears that when the jetties at the entrance are reconstructed and the shoaled area has been removed, outbound currents of slight to moderate velocity should prevail in the channel most of the time. No conclusion can be formed at this time (1960) as to whether or not the entrance will be virtually self-maintained.

ACKNOWLEDGEMENT

The conclusions in this paper are based on studies made by personnel in the Engineering Division of the Galveston District and the staff of the Beach Erosion Board of the United States Army Corps of Engineers, in the design of the jetties and channel at Port Mansfield, Texas, under the civil works construction program of the Corps of Engineers. Data on the design hurricane are from publications of the USWB. The permission granted by the Chief of Engineers to publish this information is appreciated.

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PLAN FOR CLOSURE OF OLD RIVER

By George M. Cookson[1]

SYNOPSIS

The closure of Old River is the crucial feature of the project for flood control and improvement of the lower Mississippi River. This operation involves an initial rock closure, followed by a final hydraulic fill. Geological features and hydraulic characteristics require this procedure in order to provide maximum assurance of positive closure within economical limitations.

INTRODUCTION

The structures for the control of flows from the Mississippi River in the vicinity of Old River are planned primarily to prevent the Mississippi River from abandoning its present channel below Old River and adopting the course of the Atchafalaya River. The primary intention is to prevent the change of course of the Mississippi River by the interposition of safe and stable structures and by the control of flows to suitable volumes; and, with these same flows, to enlarge and maintain at no more than a safe value, the flood-carrying capacity of the Atchafalaya River.

The authorized project for Old River control provides for construction of a controlled diversion system north of Old River, completion of the right bank Mississippi River levee system in the vicinity of Old River to full grade, and a closure dam in Old River. The new diversion system consists of a low-sill structure in the levee line with appropriate inflow and outflow channels to control flows below bankfull stage, and an overbank control structure to be operated for the diversion of excess flood flows from the Mississippi River. A

Note.—Discussion open until February 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 3, September, 1960,

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navigation lock and connecting channels are under construction on the south bank of Old River, adjacent to the closure, to provide continuation of existing commerce.

LOCATION AND DESCRIPTION

The junction of Old River with the Mississippi is approximately 302 miles above Head of Passes. It is at the latitude of the Louisiana-Mississippi State line, (Fig. 1), about 60 miles northwest of Baton Rouge, Louisiana.

FIG. 1.—VICINITY MAP

The closure dam in Old River is located approximately 2,100 ft downstream from the site of the former Texas and Pacific Railroad Bridge (Fig. 2). The location is about 1 1/2 miles from the junction of the Mississippi River and Old River, about 5 1/2 miles from the junction of the Red, Atchafalaya, and Old Rivers, and about 6 1/2 miles southwest of the low-sill and overbank control structures (Fig. 3). The location of the closure is adjacent to the site of

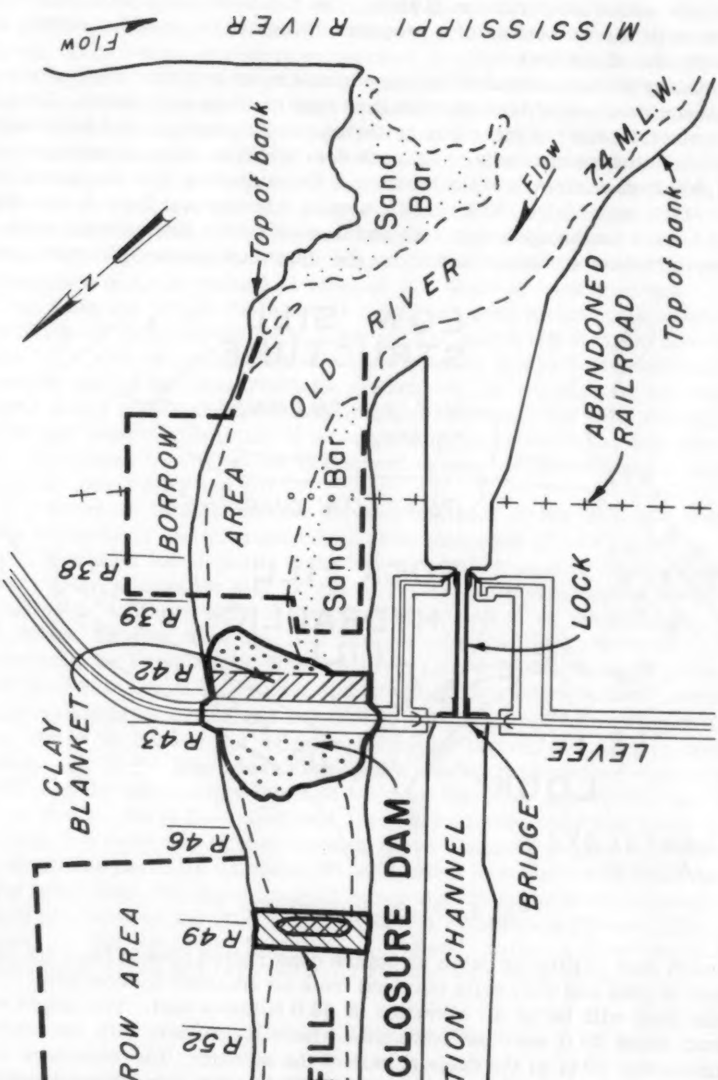

FIG. 2.—LOCATION OF CLOSURE

the lock now under construction (1960). The final, or main dam will be in alignment with the centerline of a proposed bridge to be provided across the downstream end of the lock.

Preliminary studies indicated that the closure must be constructed of a material that could withstand the high velocities that could be anticipated. Locally available material did not meet this criterion. Accordingly, it was determined that an initial closure consisting of a rock dam would be constructed approximately 1,900 ft downstream of the location of the main dam, to a minimum elevation of +17 ft msl with a 30 ft crown sloping downstream 1-on-3, and with slopes of 1-on-1 on the upstream side and 1-on-2 on the downstream side. A graded seal blanket would be added on the upstream slope of the rock dam.

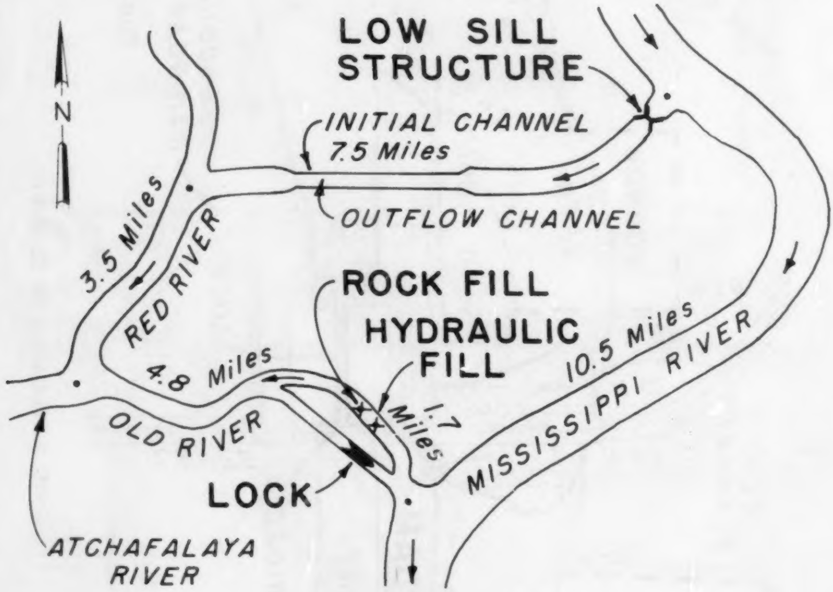

FIG. 3.—PLAN OF DIVERSION

The main dam is to be an earth structure constructed of sand from the river bed and of clay and silty soils obtained from an adjacent borrow area. The top of the dam will be at an elevation of 68.0 ft above msl. The height will range from about 20 ft near the river bank where it connects with the levees, to approximately 99 ft at the deepest part of the stream. The structure will be nearly 1,450 ft long with a crown width of 60 ft. The side slopes from the top of the dam down to an elevation of +37 ft msl will be 1-on-4. Below the elevation +37 msl the side slopes will be approximately 1-on-20 for a distance of 500 ft from the centerline, then approximately 1-on-6 down to the bottom of the river.

GEOLOGY AND SOILS

Located within the alluvial valley of the Mississippi River, the geologic history of Old River is an integral part of the history of the Mississippi. As a

result of cyclic glaciation and variations in sea level during the Pleistocene epoch, the Mississippi River in the vicinity of Old River and elsewhere in the Alluvial Valley became deeply entrenched in older sedimentary beds. Later, the entrenched valley was partially refilled and buried with sediments that were washed into the entrenchment by numerous tributary streams and graded by the river. During the early stage of valley refilling, the Mississippi transported and deposited relatively coarse materials as the aggrading valley surface developed. As the deposit in the entrenched valley thickened, the stream gradient was reduced, and the alluvium that was carried by the river and deposited became finer. The river changed from a braided to a meandering stream with systematic channel migration that resulted in the formation of conspicuous meander belts and adjacent low lands or back-swamps.

The material filling the alluvial valley has been divided into a substratum consisting of clean sands with some gravel, and a top stratum consisting of clays, silts, and silty sands. Based on the manner in which the sediments were deposited and on the characteristic gradations, the top stratum has been subdivided into a point bar deposit consisting predominately of silty sands, a channel filling deposit consisting of lean clays and silty sands, a back-swamp deposit composed mostly of fat clays, and a natural levee deposit composed mostly of sandy silts and silt.

Old River is located across the western half of the meander belt of the present course of the Mississippi, an area composed of natural levee, channel filling, and point bar deposits underlain by a substratum of clean sand. Prior to 1831 a large meander loop of the Mississippi River occupied the area, and the lower or southern limb of the loop was located along the present course of Old River. At that time the Red and Atchafalaya Rivers were directly connected with the Mississippi. The Red River joined the Mississippi at the northwest section of the meander loop, and two miles downstream from this junction the Atchafalaya River formed and flowed away from the Mississippi. In 1831, Capt. Henry M. Shreve cut a channel through the narrow neck of land at the eastern end of the loop which expedited a natural process and caused most of the flow in the Mississippi River to by-pass the loop and follow the shorter cut-off route. Some flow continued through both the upper and lower limbs of the loop, but reduction in flow through these channels caused sediments to be deposited and partially fill them. Finally, aided by some dredging at the mouth of the lower limb, the upper limb silted-up and the lower limb became the only connection between the Mississippi, Red, and Atchafalaya Rivers. The present channel of Old River is, therefore, a channel, within a large channel of the Mississippi, which has been partially refilled with sediments.

FIELD EXPLORATIONS

General-type borings were made in Old River and along its banks, in the area where the closure is located. These borings extended to elevations of -60 ft to -120 ft msl. Driving resistances on a split spoon sampling device were obtained on a representative number of borings penetrating the sand strata. Two 5-in. diam undisturbed borings were made in the area of the final closure to elevations -68 and -56, respectively, and undisturbed samples were obtained from the clay strata in the foundation. General type borings were

made in the sand bar at the entrance to Old River, opposite Carr Point revetment, and in the sand bar on the east side of the Mississippi River opposite the entrance to Old River. These borings extended to elevations -65 ft to -70 ft msl. General type borings extending to elevations +24 to +30 were made in the borrow area on the north side of Old River adjacent to the final closure site.

SOIL CONDITIONS

The banks of Old River in the vicinity of the closure site consist generally of a surface layer of sandy silt and silty sand 5 ft to 20 ft thick overlying a layer composed of lean and fat clays ranging from 20 ft to 50 ft thick, which is underlain by fine sand. There is a deposit, predominately of clay, varying from 20 ft to 40 ft thick lining the bed of the river extending from bank to bank in the vicinity of hydrographic range 49, (Fig. 2). The clay deposit extends downstream about 900 ft and approximately the same distance upstream. Progressing further upstream the clay in the river bed becomes thinner and, at a point 2,700 ft upstream from range 49 and continuing towards the Mississippi, there is no clay, as disclosed by the borings taken in this reach. It was noted, as disclosed by borings, that the clay at the bottom of the river, as shown by the sections of ranges 39 and 49, is not continuous between these ranges. A fine grained sand bar centered on range 39 extends for a distance of about 3,000 ft. Driving resistances, measured with a standard split spoon sampler, indicate that the sand in Old River varies from loose, where it is exposed, to dense and very dense at elevation about -60 ft msl. The bar at the entrance to Old River opposite Carr Point Revetment and the bar on the east side of the Mississippi River opposite the entrance to Old River consist of fine sand.

The borrow area on the north side of Old River, adjacent to the closure area, is predominately fat clay to a depth of about 20 ft below ground surface. Beneath this surface clay stratum there is generally a stratum composed of sandy silt and silty sand which changes gradually with depth, to fine sand at elevations varying between -3 and +25. Figs. 4 and 5 show sections of the initial and final closures and the results of borings at these locations.

LABORATORY TESTS

Visual classification and water content determinations were performed on samples from all borings. Sieve analyses were made on typical sand samples and representative gradation curves of these findings were plotted. Permeability tests were run on composite sand samples from borings that were representative of the sand to be used in the main closure. Consolidation, unconfined compression, and shear tests were made on typical undisturbed clay samples from borings made in the area of the final closure. Unconfined compression tests were also conducted on small core samples of the clays obtained from general type borings.

HYDRAULICS

The direction and quantity of flow through Old River is influenced by the stage in the Mississippi River at the mouth of Old River, and the stage at the

junction of the Red, Atchafalaya, and Old Rivers. Prior to 1916, flow in Old River was alternately to and from the Mississippi, but since that time the number of days of flow toward the Mississippi River has gradually decreased, so that since 1942, with the exception of 9 days in 1945, the flow has been from the Mississippi River to the Atchafalaya River. Prior to the final closure the overbank structure, the low-sill structure, and the connecting channels will be completed. Remaining to be completed (as of 1960) before Old River can be closed is a channel connecting the Mississippi River with the low-sill structure, and the levees along the Mississippi River to the ends of the closure dam.

DIVERSION OF FLOW

In the process of closing Old River, the low-sill structure will be opened. The flow diverted from the Mississippi River to the Red River through the

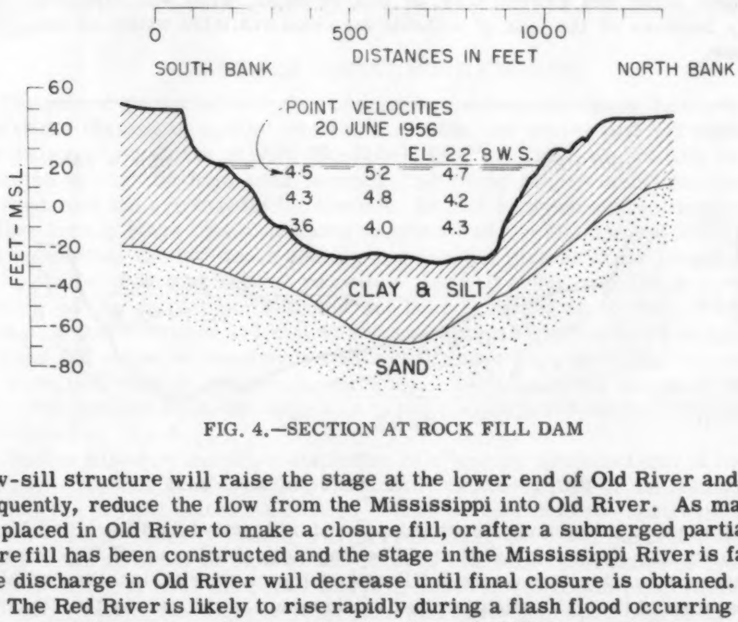

FIG. 4.—SECTION AT ROCK FILL DAM

low-sill structure will raise the stage at the lower end of Old River and, consequently, reduce the flow from the Mississippi into Old River. As material is placed in Old River to make a closure fill, or after a submerged partial closure fill has been constructed and the stage in the Mississippi River is falling, the discharge in Old River will decrease until final closure is obtained.

The Red River is likely to rise rapidly during a flash flood occurring at any season of the year and reduce the flow through Old River. It does not follow as regular a seasonal discharge sequence as the Mississippi River. Although some flow from the Red River can be anticipated during the season when the Mississippi River is at a stage of about +45 (by chance this flow could be large enough to advantageously alter the stage at the mouth of Red River), the quantity that can be expected from the Red River with reasonable assurance will not exceed 15,000 to 20,000 cfs. This would raise the tailwater of the closure dam less than 1.0 ft. Therefore, the flow from Red River was not included in the study for either highwater or low water conditions of flow. The use of the lock to divert flow is not contemplated, but the culverts in the lock walls could be used for short durations in emergencies. Since the soils comprising the

lower part of the lock channels are easy to erode, prolonged flows with relatively high velocity through the channel would result in serious scouring.

DESIGN OF CLOSURE

Initial consideration was given to a plan whereby a fill would be constructed of successive layers of sand, gravel, and rock, each to the limit of its stability under prevailing velocities. This would be done on the alignment of the levees and of the bridge over the lock. Involved, however, would be the difficulty of controlling the placement of materials to the extent necessary to prevent a crevasse from occurring, especially while the rock is being placed. If a crevasse should occur through the rock while the closure is being made, it would extend rapidly into the finer grained gravel and sand and make it impossible to prevent the scouring out of the greater part of the fill. No further consideration was given this plan, due to the inherent risk involved.

Study was made of a plan to make a single complete dredge fill on final alignment using pea gravel, clay, or coarse sand. This was discarded primarily because of the lack of suitable material available within an economic distance.

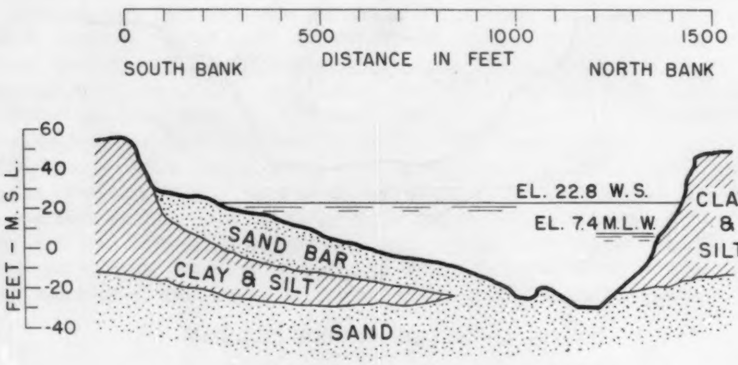

FIG. 5.—SECTION AT MAIN CLOSURE DAM

Preliminary studies and computations based on tractive, bed load, and Manning's formulae disclosed that it would be impracticable to cut off flow through Old River by constructing a single fill across the river with locally available sand because of the high velocities that would occur while making the closure. It is apparent, therefore, that before the main closure dam could be constructed the flow would have to be cut off by constructing an initial closure fill. The following methods were considered for making the initial closure:

1. Construct a clay fill during the low water season by means of a hydraulic dredge.
2. During falling stages at the end of the highwater season, construct a fill consisting of sand at the base, rock at the top, and gravel between the sand and rock.
3. Construct during the highwater season, a submerged rock fill, on lumber mattress, to an elevation that will cut off flow as the river falls at the beginning of the low water season.

4. Similar to method 3, except that concrete tetrahedrons would be used instead of rock.

5. Similar to method 3, except that layers of willow or frame crib timber mattresses would be used in conjunction with the rock for constructing the submerged dam.

Method 1 was rejected after subsurface explorations disclosed that there was not enough suitable clay available within economic reach of the closure site. Method 2 was rejected after it was disclosed that there was inherent risk of losing the fill as a result of crevasses occurring during construction, which would allow scouring through the coarse materials down into the underlying finer material. Methods 3, 4 and 5, are similar except for the materials used. Because of the comparatively high estimated cost of building and placing concrete tetrahedron blocks, this method was rejected. Although the estimated cost of the rock fill closure, method 3, was slightly more than the cost of method 5, it was selected for making the closure because it was considered the more stable and easier to construct. It would be very difficult and impractical to place the layers of willow or timber mattresses with the high velocities that prevail.

DESIGN OF INITIAL ROCK CLOSURE

The site of the initial rock dam closure was selected because the cross section of the channel is nearly symmetrical about the center line and velocities are relatively uniform at this location (Fig. 4). The rock fill will be constructed during the highwater season. The basic requirements for the submerged rock fill are that its top elevation be just high enough to insure cut off of flow for any stage condition likely to prevail as the Mississippi River stage falls preceding the low water season, and that the rock be large enough to remain stable under the highest velocities that will occur over the dam. From a study of the daily river stages which have occurred since 1931, it was determined that elevation +17 msl was the minimum elevation to which the submerged fill could be constructed with reasonable assurance that it would cut off flow long enough to provide sufficient time to complete the main closure fill. The periods when the stage was below elevation +17 during 1931 to 1956 are shown on Fig. 6.

Studies based on the stage-discharge relationship disclosed that at the time the rock fill will be completed, and during the early part of the falling stage period, the highest velocities over the rocks will occur at the upstream shoulder of the fill. This condition will prevail until the river falls to a stage at which critical depth of flow at the crest is reached. Thereafter a hydraulic jump will occur over the rocks and will move downstream progressively as the stage falls. The highest velocities will occur just upstream of the jump. For the size and shape of fill selected, the computed velocities of flow over the rock fill for various stages in the Mississippi River or various discharges are shown on Fig. 7. The quantity of water that will flow through the rock fill was not included in the analysis of the flow over the rock fill and in determining the stability of the rocks in the fill. It was determined, by computations, that the mean velocity at the crest of the dam will vary from about 9 fps at a stage of about +45, to 15 fps at a stage of about +30. The mean velocity just upstream of the jump will vary from about 15 fps at a stage of +30 to a maximum of 20 fps to 22 fps at a stage of about +24. The required rock size was based on Isbash's formula, modified to provide correction for slope.

For a maximum velocity of 22 fps, and at a unit weight of 115 lbs per cu ft for the rock, the required stone weight is 4,200 lbs. Therefore, derrick, or

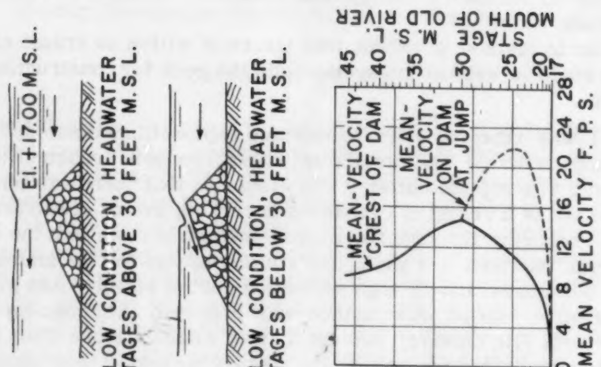

FIG. 7.—FLOW CHARACTERISTICS OVER ROCK FILL

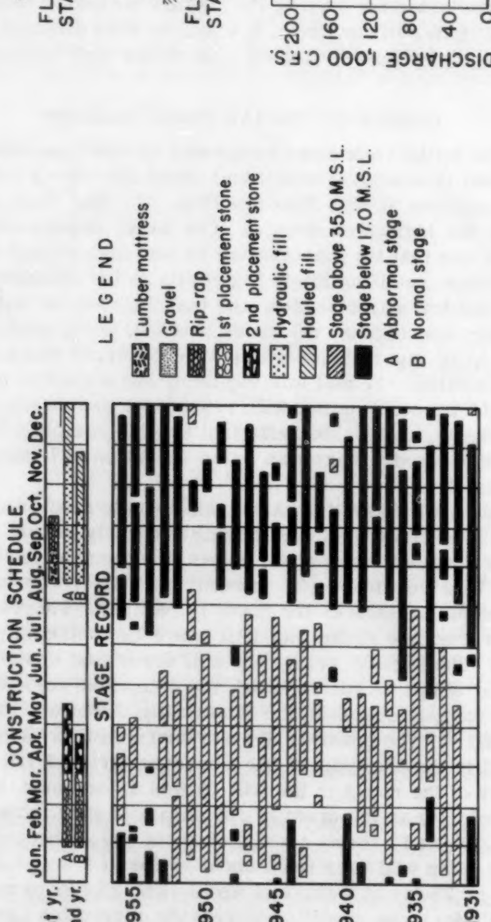

FIG. 6.—CONSTRUCTION SCHEDULE AND RELATION TO EXPERIENCED STAGES

jetty stone, of which at least 65% of the individual stones in each barge load shall be 2 1/4 tons or heavier in weight will be used to make the rock closure. The side slopes of the rock fill will be 1-on-1.25 on the upstream side and 1-on-2 on the downstream side. The stability of the proposed rock fill was tested in a model study at the United States Army Corps of Engineers Waterways Experiment Station at Vicksburg, Mississippi. Based on this study, the proposed slopes are considered necessary to provide a reasonable factor of safety. The study disclosed that a seal blanket on the upstream slope of the rock fill would be necessary to prevent loss of final closure material through the rock. As a result, a seal blanket consisting of a layer of riprap 4 ft thick and a layer of gravel or shell 1.5 ft thick will be placed on the upstream slope of the rock fill, plus an impervious blanket to be pumped and discharged over the upstream face of the fill at the time construction of the main closure dam is commenced. The gradation of the riprap and shell or gravel will be as indicated in Table 1.

TABLE 1.—GRADATION OF UPSTREAM SEAL BLANKET

Size, Inches	Per Cent Passing, by Weight
Riprap (Coarse Layer)	
12	100
7 1/2	60 - 90
5	10 - 40
2 1/2	2
Gravel or Shell (Finer Layer)	
2 1/2	100
1	60 - 90
1/2	30 - 60
1/4	10 - 35
1/20	10

To prevent erosion of the soils in the river bed at the base and downstream of the rock fill, and in the river banks above the elevation of the crest of the rock fill, a lumber mattress and a gravel blanket will be placed over the river bed and on the banks, respectively, as shown on Figs. 8 and 9. A layer of riprap 2 ft thick will be placed over part of the lumber mattress and over the gravel blanket to add additional protection against erosion.

The derrick stone fill will consist of two placements. The first placement will be 5 ft to 6 ft thick, extending up the slopes of the banks from approximate elevations -5 to +35 msl (Figs. 8 and 9). The second placement of derrick stone will be placed over the riprap fill and will be accomplished with a minimum of 18 ft difference between the headwater stage and the elevation of the top of the rock being placed in order to avoid placing stone in high velocity currents. It would have been desirable to locate the rock dam so that it could be incorporated in the downstream portion of the main closure dam, but this was not possible because of hydraulic requirements of the initial closure and the required alignment of the main closure with the bridge across the navigation lock.

DESIGN OF MAIN CLOSURE

The proposed cross section for the main closure dam is shown on Fig. 10. The lower portion of the fill will be composed of sand and the upper part of

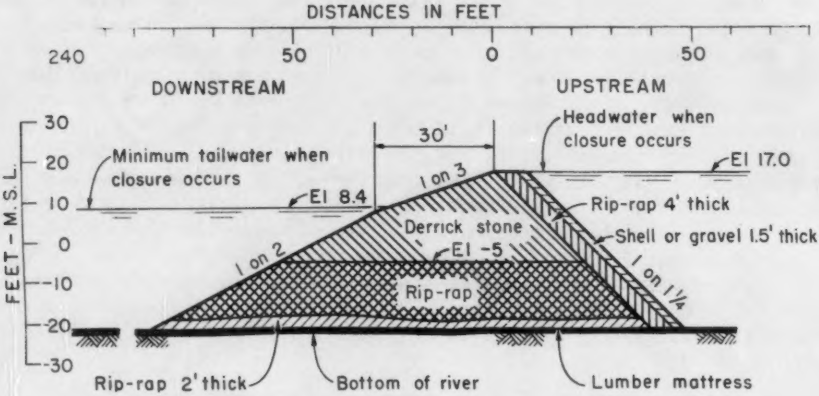

FIG. 8.—CROSS SECTION OF ROCK FILL DAM

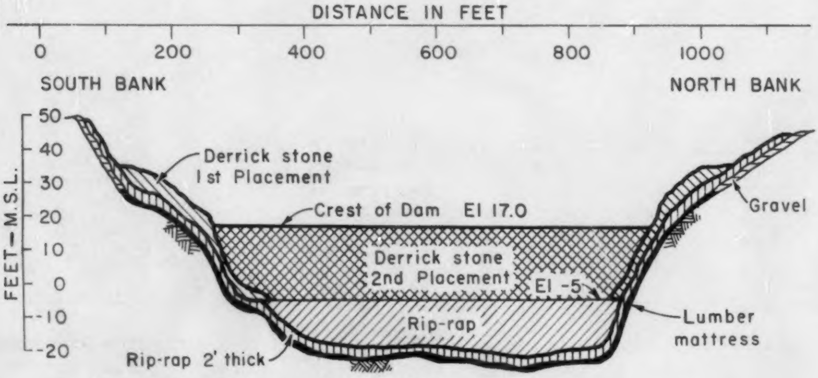

FIG. 9.—PROFILE ALONG CREST OF ROCK FILL DAM

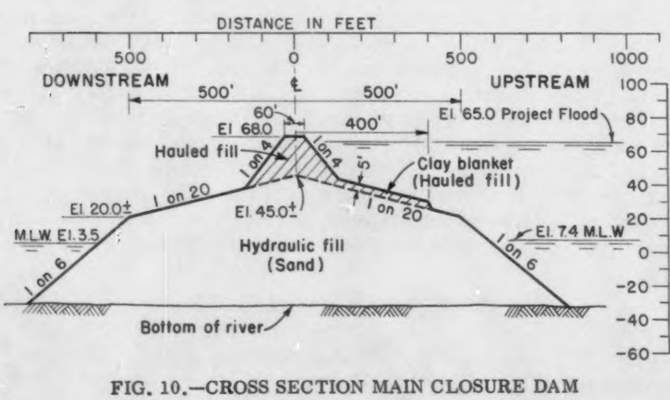

FIG. 10.—CROSS SECTION MAIN CLOSURE DAM

clays. The 1-on-6 slope for the sand fill below water surface and the 1-on-20 slope above the water surface were based on the natural angle of repose of a previous hydraulic sand fill made across Old River at its entrance with sand obtained from the bed of Old River. The remainder of the main closure fill will consist of clay hauled from an adjacent borrow pit and compacted into a section with 1-on-4 side slopes and a crown 60 ft wide. The exact height to which the sand portion of the fill will be constructed will depend upon the river stage at the time it is placed. A blanket of clay, 5 ft thick, will be placed on the upstream side of the closure over the sand fill exposed above water, and over the sand bar for a distance of 400 ft upstream from the closure center line, for the purpose of reducing seepage through the embankment. A portion of the completed dam along the south one third of the river crossing will be located over a clay stratum which extends in depth to approximate elevation -50 msl.

Based on an assumed failure to the bottom of this clay and using the design shear strength in the clay, and the seepage influence indicated by the flow net, the proposed dam has a minimum factor of safety of 1.5. Assuming a friction angle of 25° with sudden drawdown to elevation +3.5 within the 1-on-6 slope of the sand fill on the downstream side, the factor of safety with respect to shear failure is 1.2. This factor of safety is considered ample because the drawdown conditions assumed are extreme.

SETTLEMENT

It is estimated, based on the consolidation curves, that 2.1 ft of settlement will occur due to consolidation of the foundation. Of this amount 0.8 ft will occur during construction and 1.3 ft after construction. Allowing 1.5 ft for shrinkage and consolidation of the fill after construction the estimated crown settlement after construction will be 2.8 ft. The net grade of the adjacent main-line Mississippi River levee is +64.4 msl. The main closure dam will be built to elevation +68. This allows 3.6 ft for settlement and shrinking.

SEEPAGE

The sand portion of the embankment will not be completely covered with the impervious blanket on the upstream side, and the base of the embankment will be in contact with the substratum sand as disclosed by the borings. Seepage will therefore occur through the exposed sand portion of the fill and through the substratum sand beneath it, as shown by the flow net.

The flow net was based on a permeability of 50×10^{-4} cm per sec for the hydraulic fill sand; horizontal and vertical permeabilities of 400×10^{-4} and 100×10^{-4} cm per sec, respectively, for the substratum sand between elevations -35 and -75, and $1{,}000 \times 10^{-4}$ and 250×10^{-4} cm per sec, respectively, for the substation sand below elevation -75.

The permeability of the sand fill was determined from the results of permeability tests on composite sand samples from the proposed borrow area, and the permeability of the substratum sand was based on the relationship between permeability and grain size obtained from field pumping tests conducted

at Old River lock site. The maximum amount of seepage that may occur on the downstream side of the closure fill, as disclosed by the flow net will be about 559 cu ft per day per linear ft of embankment. Seepage in this quantity could occur only at high river stage when the embankment is first completed, and will progressively decrease in time as fine sediments from the Mississippi River blanket the upstream side of the dam.

BORROW PITS

Materials for the hydraulic fill portion of the main closure will be obtained from the sand bar on the south bank of Old River and from the river bottom on the upstream side of the closure. If required, additional borrow areas on the north bank of Old River will be available, as shown on Fig. 2. The borrow pits for the hydraulic fill will be located a minimum distance of 500 ft from the upstream toe of the main closure fill. Material for constructing the upstream impervious blanket will be obtained from a borrow area located adjacent to the closure on the north side of Old River, as shown on Fig. 2. It is contemplated that materials excavated from the channels for the proposed navigation lock on the south bank of Old River adjacent to the closure will not be available for making the closure. However, should the closure be constructed prior to excavation of the channels, suitable materials from the channel areas will be used insofar as practicable.

Riprap and stone are not available locally and will have to be obtained elsewhere and transported to the site of the work in barges.

METHODS OF CONSTRUCTION

Closure of Old River will be accomplished in three phases of construction; preparation of the initial closure area to prevent erosion, construction of the initial closure (rock fill dam), and construction of the main closure (hydraulic fill and hauled fill).

The dumping of all materials in the water will be controlled so that the materials will settle as near as possible within the predetermined location on the river bottom. Control will be exercised by dumping the materials at a trial distance upstream from the desired location, depending on the velocity of the water and depth of the river. Soundings will be taken to determine the actual location of the dumped material on the bottom of the river, then the distance upstream to the dumping point will be adjusted accordingly.

Preparation Of Initial Closure Area (Rock Fill Dam).—During the low water season in the year preceding the construction of the main closure, an area consisting of a strip 300 ft wide extending across Old River will be prepared as a base for the initial rock closure. As shown on Fig. 2, the upstream boundary of the area will be 60 ft upstream from hydrographic range 49 and the downstream boundary will be 240 ft downstream from this range.

The upper banks of Old River within the base area will be graded to a 1-on-3 or flatter slope. A continuous lumber mattress will be placed over the base area below water. The upper banks above water elevation will be paved with a 12-in. layer of gravel covered with 24-in. of riprap. The portion of the revetted area 40 ft upstream and 90 ft downstream from hydrographic range 49 and below the water surface extending across Old River, will be covered

with 24-in. of riprap to form a base upon which to place the riprap and larger stone. The riprap will be delivered to the site on barges and will be handled from barges by draglines at a minimum rate of 1,000 tons per day. The rate is based on unloading two barges at a time by dragline. Estimated time required to complete the preparation of the base is 50 days. This work is scheduled to be done during the months of August and September of the first construction year, but may be done in any suitable period, preceding placement of stone.

Construction Of Initial Closure (Rock Fill Dam).—The initial closure will be accomplished by placements of riprap and stone during the high water season following the preparation of the base (Figs. 8 and 9). Placement of the stone will be interrupted or discontinued when the difference between the headwater elevation and the elevation of the top of the stone being placed is 18 ft or less, as would have occurred in the years 1931 and 1954 (Figs. 11 and 12). The first placement is scheduled to insure completion by the 23rd of March,

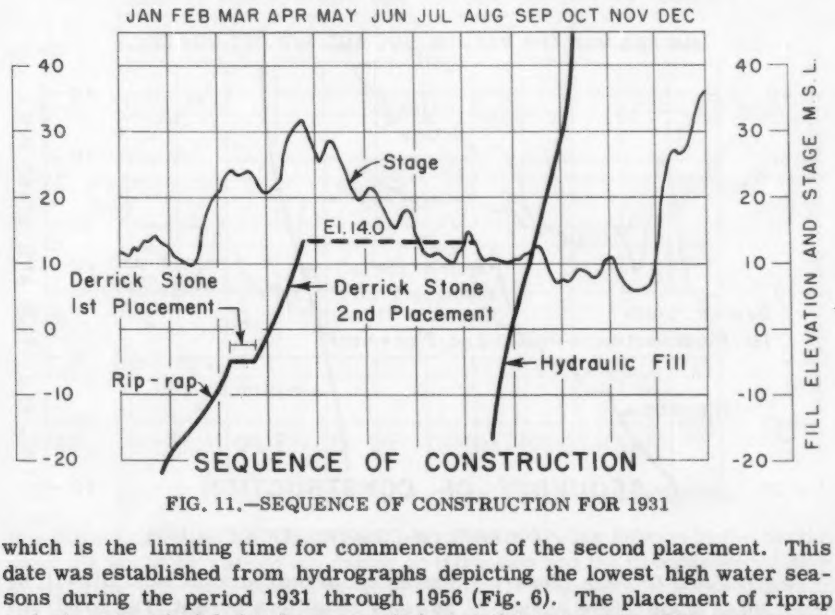

FIG. 11.—SEQUENCE OF CONSTRUCTION FOR 1931

which is the limiting time for commencement of the second placement. This date was established from hydrographs depicting the lowest high water seasons during the period 1931 through 1956 (Fig. 6). The placement of riprap and stone is scheduled to be accomplished at the same rate. The commencement date of the riprap placement may be advanced, however, if desired to January 1 and the placement rate reduced accordingly. The first and second placements of stone will be made as scheduled or faster. The seal blanket will be constructed when the head over the rock fill dam has been reduced to about 3 ft and a continuous fall in stage is predicted.

The riprap will be placed in horizontal layers 3 ft to 4 ft thick to elevation -5 msl. The first placement of derrick stone will consist of a layer 5 ft to 6 ft thick of 2 1/4 ton stone extending up each bank from elevation -5 to elevation +35 msl (Fig. 9). The second placement will be accomplished by progressively building up the cross section above the riprap in horizontal layers about one rock thick to attain an upstream slope of 1-on-1.25, a crown slope of 1-on-3,

and a downstream slope of 1-on-2 (Fig. 8). Placing of the 2 1/4 ton stone will continue until the initial rock fill is completed to elevation +17 msl or to the maximum elevation reached before termination of construction due to the limiting depth of water flowing over the rock. A review of the hydrographs, 1931-1956, shows that the rock dam could have been constructed to elevation +17 msl in 19 of the 26 yr. In the other 7 yr the initial closure could not have been constructed to elevation +17, but the low water following the placement of the stone would have been low enough to insure final closure.

To allow for settlement, and stone and riprap placed out of section, the quantities of stone and riprap required were computed on the basis of 1.75 tons per cu yd rather than 1.35 tons normally used for dry land construction. In addition, the quantities were increased by about 13 1/2% to allow for other contingencies. The riprap and 2 1/4 ton derrick stone will be delivered to the site on barges and placed in position in the river at a minimum rate of 1,700 tons per day. Estimated time required to place the stone for the initial closure is 86 days.

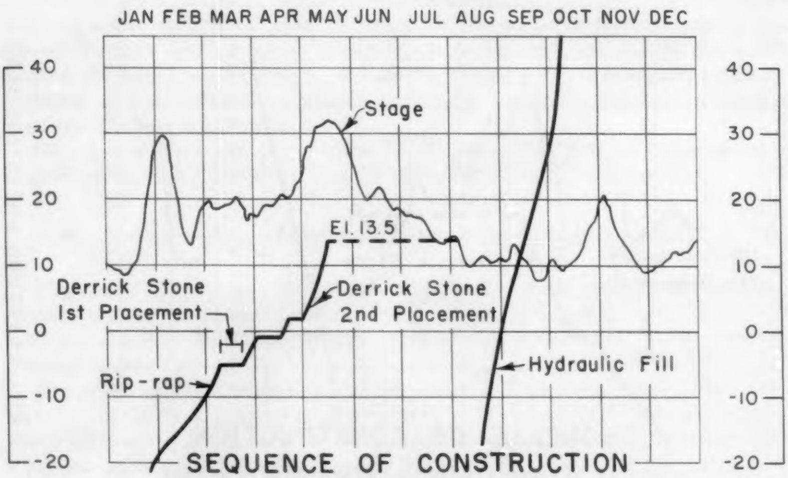

FIG. 12.—SEQUENCE OF CONSTRUCTION FOR 1954

Construction Of Main Closure.— Placement of the hydraulic sand fill will be commenced on or before August 15, as river conditions permit. The construction schedule shown (Fig. 6) contemplates commencement on August 15. At that time, based on past records, flow in Old River will either be cut of completely by the rock fill or it will be reduced sufficiently to permit the placement of sand without appreciable loss. The sand will be placed at a rate that will insure completion of the portion of the closure cross section up to elevation 0 msl in 15 days, to elevation + 28 in 45 days, and up to elevation +35 by October 6.

Based on the assumption that 50% of the material pumped by the dredges will remain in the planned cross section, and an average rate of pumping for one dredge (including allowance for breakdowns) of 35,000 cu yd per day, two dredges will be required to accomplish the hydraulic fill work according to

the afore-mentioned schedule. The relationship of the sequence of construction of the hydraulic fill to average hydrographs is shown on Fig. 13.

A study of actual hydrographs shows that in most cases (22 yr out of 26 yr) the schedule could have been followed without interruption. The year 1950 (Fig. 14) represents the worst of four cases where the schedule could be disrupted and delay could result because of prevailing abnormal high stages during the months of September and October. The year 1945 (Fig. 15) represents the only case in which a rapid rise and abnormal high water during the low water season would have almost overtopped the fill.

Based on the records of the past 26 yr (since 1933), the construction sequence shown on Fig. 6 allows sufficient time to complete the hydraulic sand portion of the closure fill with two dredges. It will require about 56 days for pumping the sand fill if normal conditions prevail, and 94 days if the most adverse experienced in the past reoccur at the time the fill is being pumped.

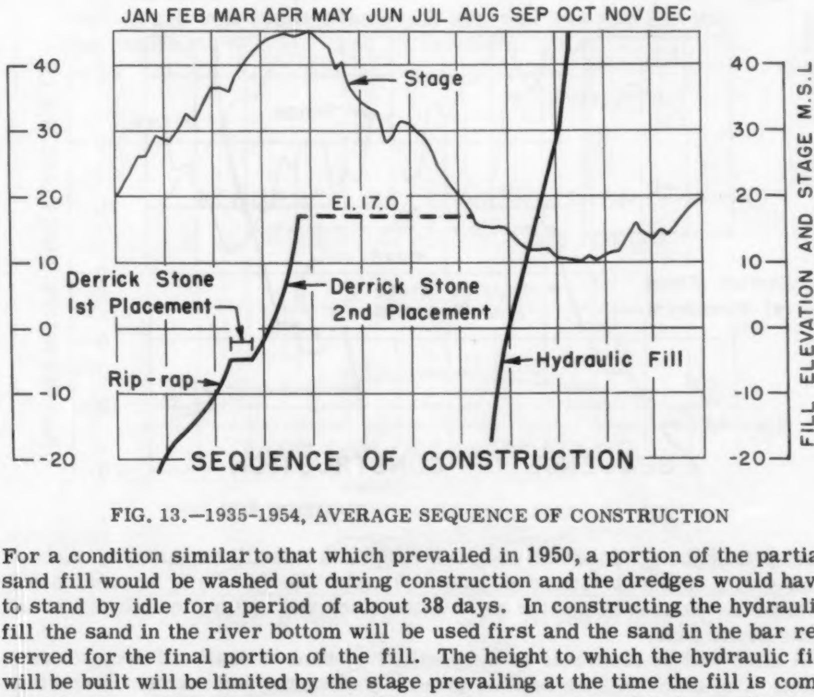

FIG. 13.—1935-1954, AVERAGE SEQUENCE OF CONSTRUCTION

For a condition similar to that which prevailed in 1950, a portion of the partial sand fill would be washed out during construction and the dredges would have to stand by idle for a period of about 38 days. In constructing the hydraulic fill the sand in the river bottom will be used first and the sand in the bar reserved for the final portion of the fill. The height to which the hydraulic fill will be built will be limited by the stage prevailing at the time the fill is completed. For estimating purposes this height was assumed to be at elevation +45.

Hauled Fill.—Immediately following the completion of the sand hydraulic fill, hauling equipment will commence placing and semi-compacting the hauled fill portion of the main closure and the impervious blanket on top of the sand fill and sand bar. The estimated time of completing the hauled fill including the impervious blanket is 45 days. An alternate plan for the hauled fill embankment is provided in the event that adverse weather or river stages make it impractical to use hauling equipment. This plan contemplates the use of an all-hydraulic fill.

The basic requirement of the alternate construction plan is to complete a stable embankment across Old River to elevation +68 msl by hydraulic method during the low water season. The fill material will be sand except for an impervious clay blanket to be placed on the upstream slope. Based on a previous hydraulic sand fill made in Old River run-off from the dredge discharge over slopes steeper than about 1-on-20 will result in serious erosion of the fill.

It is proposed to construct the sand fill in four phases as described below and as shown by the cross sections on the sketch, Fig. 16. A clay blanket will be added on the upstream side as Phase V. In Phase I, the embankment will be placed as planned in the design memorandum to about elevation +45 msl with side slopes of about 1-on-20 to water surface and 1-on-6 below water surface. In Phase II, retaining dikes located about 200 ft each side of and parallel to the embankment centerline will be constructed with sand obtained from the center of the embankment, as shown on the sketch. These retaining

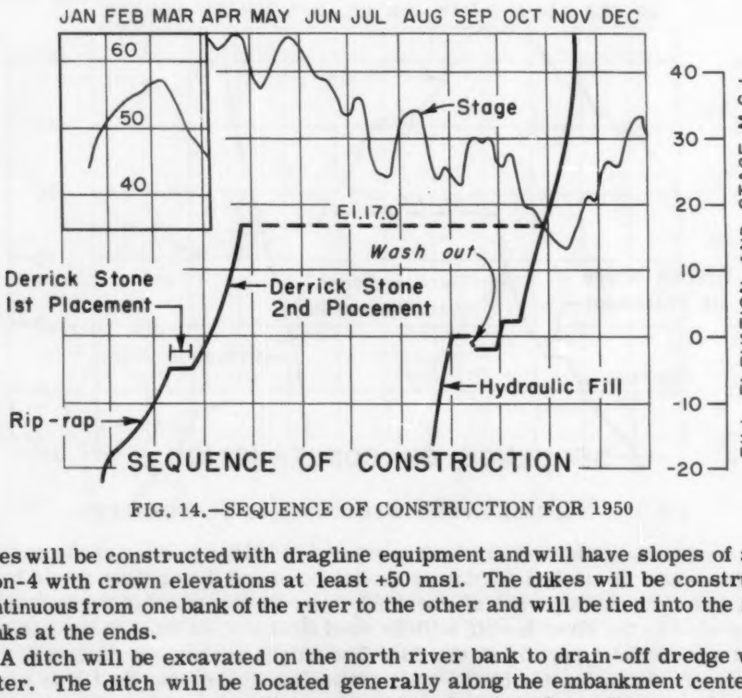

FIG. 14.—SEQUENCE OF CONSTRUCTION FOR 1950

dikes will be constructed with dragline equipment and will have slopes of about 1-on-4 with crown elevations at least +50 msl. The dikes will be constructed continuous from one bank of the river to the other and will be tied into the river banks at the ends.

A ditch will be excavated on the north river bank to drain-off dredge waste water. The ditch will be located generally along the embankment centerline where it commences and will be normal to the river for a distance about 200 ft from the river. From there, it will circle along the river bank and connect back with the river about the embankment. The bottom of the ditch will be at about elevation +42 msl, and will drain off the waste dredge water between the retaining dikes but will also permit water to be impounded on the hydraulic fill between the retaining dikes without over topping the dikes.

In Phase III, a second lift hydraulic fill will be placed between the retaining dikes. This fill will be constructed mostly in impounded water to about elevation +55 msl with ultimate slopes of 1-on-20. The waste water run-off during

construction of Phase III will either flow through the ditch in the north river bank or seep through the embankment. In either event the disposal of the waste water will not result in erosion of the fill. The hydraulic fill in Phase III will be constructed from the south bank progressively across the river.

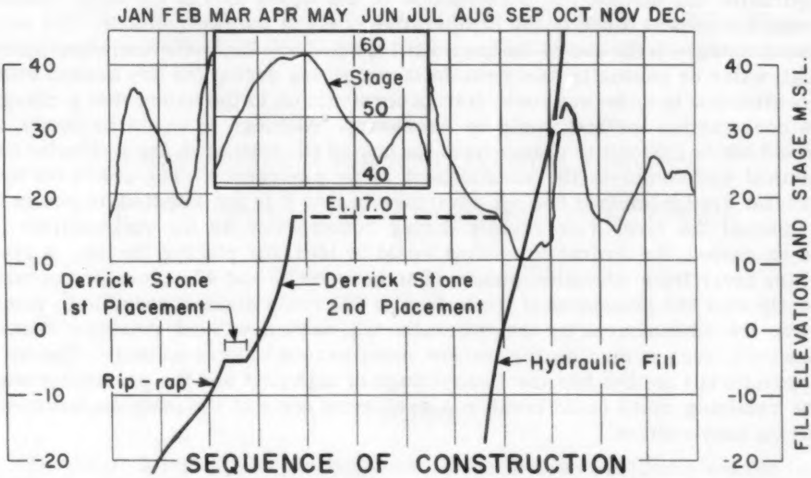

FIG. 15.—SEQUENCE OF CONSTRUCTION FOR 1945

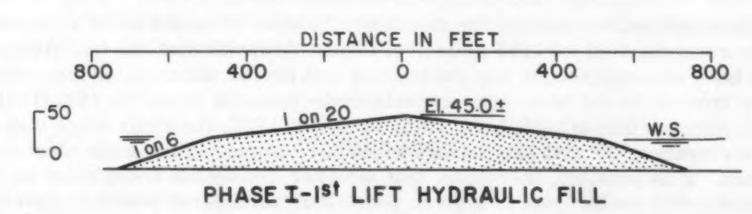

FIG. 16.—ALTERNATE HYDRAULIC CLOSURE

In Phase IV, the upper part of the embankment will be raised and shaped by dragline to a crown width of 60 ft at elevation +68 msl with side slope of 1-on-6, as shown on the sketch. The impervious blanket will be placed, in Phase V. This blanket will consist of clay obtained from the clay stratum in the north

bank of Old River between approximate elevations +25 and +50 msl during high water stage, or from the clay stratum beneath the sand bar near the south bank during low water. A 20-in., or smaller, dredge will be used for placing the blanket. Use of a larger dredge will wash away the sand fill.

The use of the hauled fill method proposed in the basic design in lieu of the hydraulic fill method for construction of the upper part of the main closure embankment was based on the comparative costs of the two methods. The only disadvantages to the use of the hauled fill method are that extreme, abnormally high water or unusually heavy rainfalls occurring during the dry season when construction is underway could disrupt construction to the extent that a charge in construction method would be necessary. Although it would be costly, it would not be difficult to change from the hauled fill method to the hydraulic fill method and complete the embankment in an emergency. The use of the hydraulic dredge method has the advantage in that it is not affected by weather.

Should the river rise rapidly during construction as the embankment is being raised, the hydraulic method would be ideal for placing the fill. A rise in the river from elevation about +17 to between 35 and 40 occurring concurrently with the placement of the hydraulic fill would make it possible to complete the embankment by the hydraulic fill method without retaining dikes. However, such a rise during the low water season is very unlikely. The all-hydraulic fill method has the disadvantage of high cost and the possibility that the retaining dikes could break and wash away some of the main embankment during construction.

CONCLUSION

The construction plan for the Old River closure is based on 26 yr of river stage records, 1931 to 1956 inclusive. These indicate that the closure could have been accomplished in any year during this period, although, in four cases, some trouble would have been experienced. Records prior to 1931 (1871 to 1930), indicate that in 1905, 1906, 1915, 1926, and 1927, the river stage was not below elevation +17 msl long enough to permit completing the main closure as planned. It is possible, therefore, that adverse conditions could exist so that closure could not be made in a given year. It is considered practical, preferable, and sound engineering practice to follow a plan that will assure against all conceivable possibilities with flexibility where dictated. Recognition should be given to the fact that any plan for a river closure must take full account of the geology and soils of the area and the hydraulic characteristics of the stream in the development of the proper engineering and economic solution.

Construction of the Old River Control Project is being conducted under the direction of the President, Mississippi River Commission, and the general supervision of the author.

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DESIGN OF INLETS FOR TEXAS COASTAL FISHERIES

By H. P. Carothers,[1] F. ASCE, and Homer C. Innis,[2] F. ASCE

SYNOPSIS

The basic formulations for dynamic balance in design of Texas coastal inlets to permit fish passage through the littoral barriers and induce Gulf water interchange for control of bay salinities are presented. Rollover and Yarborough passes illustrate extremes of erosion and siltation. Cedar Bayou illustrates elastic limit design toward siltation.

INTRODUCTION

There is a definite need for additional inlets—locally called fish passes—through the barrier islands of the Texas coast, for the improvement of the coastal fisheries. Since some sport fish spawn in the Gulf and move to the bays, while others spawn in the bays and move to the Gulf, provision for the free movement of fish through such inlets is being made to improve the fishing.

The bays in the humid eastern region often tend to become too fresh or too low in salinity, while those in the semi-arid southern region of the coast often become hypersaline and starved for Gulf water interchange. Reasonable control of these bay salinities is another important function of artificial coastal inlets, or fish passes.

It is the design intent to provide, so far as possible, natural inlet operation of these passes, eliminating maintenance dredging and stabilization works.

Note.—Discussion open until February 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 3, September, 1960.

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The basic design procedure includes formulas or methods for analyzing tidal differentials, size of littoral material, median design velocity, littoral transport intercepted from both longshore directions, sediment transport, use of the Gulf bar as the automatic mechanism for dynamic balance, and salinity currents and salinity control in the bays.

The locations of the various passes along coastal Texas are shown on Fig. 1. These inlets are the only connections through the barrier reefs between the Gulf of Mexico and the shallow coastal bays.

Rollover Fish Pass, near Galveston, is used to illustrate a short, high velocity pass which, prior to extensive stabilization, eroded considerably. Water interchange is needed here to control low salinities. Littoral transport is predominantly westward, with corresponding downdrift beach erosion.

Yarborough Pass, now closed, is used to illustrate the other extreme of complete imbalance on the side of extreme siltation, in which the pass was very rapidly overwhelmed by the littoral transport.

Cedar Bayou Fish Pass illustrates a small, natural intermittent pass with the design extended to the elastic limit or yield point toward siltation.

A proposed Boggy Slough Fish Pass illustrates a large water interchange pass for control of hypersalinity in the semi-arid region of the southern coast, and a proposed Corpus Christi Fish Pass illustrates a balanced design.

DESIGN INTENT AND PROCEDURE

The records of the Texas Game and Fish Commission show that various attempts to open or maintain fish passes were begun in 1939. These attempts—largely unsuccessful and undertaken without application of tidal hydraulics—were made by dredging at the Old Corpus Christi Pass, Cedar Bayou Pass, Yarborough Pass, and Rollover Pass (see Fig. 1 for location of passes). In 1952, serious engineering studies were begun, to develop a proper design procedure for these inlets (fish passes) through the barrier islands.

In 1957, tidal and beach erosion data were secured at Rollover Pass by the U. S. Beach Erosion Board. In 1958, the Texas Game and Fish Commission secured considerable basic data at Cedar Bayou and, finally, in 1959, sufficient data were secured at one locality, the Upper Laguna Madre, to permit completion of a basic design procedure for fish passes.

Several of these passes along coastal Texas (shown on Fig. 1), including the unsuccessful attempts, serve at least as a full-scale, wide-range experimental program on which to base the development of the design procedure presented herein. It now appears quite probable that the design intent of providing an inlet that operates naturally, or with a minimum of maintenance dredging and stabilization works, can be approached by application of the following basic procedure and formulation:

1. Secure and analyze hourly tidal differentials across the barrier island;
2. Compute the first approximation of potential cross-sectional area of the inlet and potential water interchange;
3. Sample and determine size distribution of the littoral material;
4. Make the first trial balance in design by sizing the inlet channel so that the velocity estimated from the median differential is equal to the design velocity;
5. Adjust the tidal differentials where appreciable change in bay tides is expected, then rebalance;

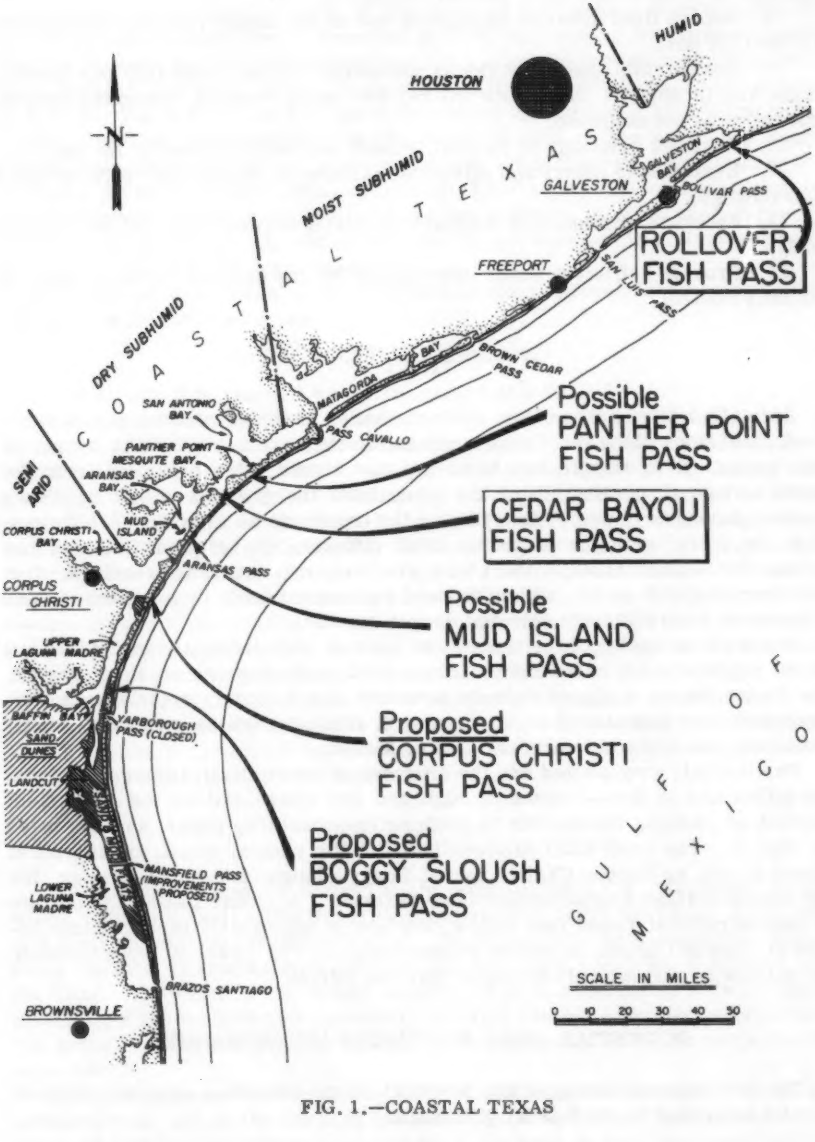

FIG. 1.—COASTAL TEXAS

6. Estimate from wave energy the total littoral transport intercepted from both longshore directions by tidal flow through the inlet;

7. Rebalance the design to equate sediment transport capacity of inlet channel with the intercepted littoral transport;

8. Secure final dynamic balance by use of the highly variable mechanism of the Gulf bar;

9. Design stabilization works as necessary to control the Gulf bar mechanism and to support imbalance where, for many reasons, complete natural operation is not achieved;

10. Evaluate relations of rainfall, runoff, and evaporation for the bay;

11. Evaluate the nature and effects of salinity or density currents on water interchange;

12. Estimate efficiency of mechanical mixing between Gulf and bay waters; and

13. Provide sufficient water interchange for the desired or most feasible salinity control.

TIDES

A detailed description of the characteristics of the astronomical and meteorological tides along the Texas coast and in the bays is beyond the scope of this paper. It is significant, however, that tropic tides resulting from the north and south declination of the moon have the greatest effect on Texas coastal harmonic tides. The cycle of the tropic month is about 27-1/3 days. The frequency analyses of hourly tidal differentials between Gulf and bay across the barrier islands, therefore, are made on continuous readings of at least one tropical month, with additional refinement made by analyses of continuous, or intermittently selected, tropic months.

Scientific sampling of the tides must include knowledge and consideration of the relative value of the many various tidal producing forces. For instance, the Cedar Bayou analyses include a winter and summer tropic month, with representative samples of strong northers, fresh prevailing southeast winds, calm periods, and a storm in the Gulf of Mexico.

Particularly emphasized are the analyses of hourly tidal differentials, both for inflow and outflow—instead of high and low tides—and on the convenient method of plotting the results on arithmetic-probability paper, as illustrated by Fig. 2. The total tidal differentials for the central coast, measured at Cedar Bayou, at Corpus Christi, and at Boggy Slough, are quite similar, but the inflow-outflow characteristics are different. At Cedar Bayou, the outflow is considerably stronger than inflow. Outflow is moderately stronger than inflow at Corpus Christi, as shown subsequently on Fig. 5(a). At Boggy Slough, the outflow is only slightly stronger than the inflow.

POTENTIAL AREA AND WATER INTERCHANGE

The first approximation of the potential cross-sectional area of the inlet may be computed by the following formula:

$$A_p = 0.185\ A_s \text{ (Central Coast)} \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which A_p is the potential cross-sectional area of pass below mean sea level, in sq ft; and A_s is the surface area of connected bays at mean tide, in acres.

Eq. 1 indicates, for the central section of coastal Texas, a general relation of the pass size to the area of connected bays for reasonable potential development of tidal interchange. The formula has been developed: first, from another paper,[3] where the ratio of the area of Gulf passes in square feet to the tidal prism in acre-feet is reported at 0.44; and second, by the assumption that the reasonable potential for median tidal prism in feet of depth with properly located and sized passes is about equal to the median tidal differential, or 0.42 ft. The 0.44 times 0.42 gives the factor 0.185 used in this formula.

For the first approximation of the potential size of the proposed Boggy Slough Fish Pass, with a bay system of about 125,000 acres, this formula indicates a potential cross-sectional area of 23,100 sq ft. But consideration of the restrictions between the basins of the bay system, as well as design velocity, sediment transport, and water interchange resulted in a proposed pass with only about half the potential area.

At Rollover Fish Pass, where the median tidal differential is computed at 0.58 ft, this formula becomes:

$$A_p = 0.255\ A_s \ \text{(Rollover)} \qquad (2)$$

which is simply 0.58 times 0.44 for the factor of 0.255.

The surface area of East Galveston Bay, east of Hanna Reef (Fig. 9), is estimated at 47,000 acres. It is tentatively assumed that Rollover Fish Pass might potentially serve at least one-third and probably about one-half of this East Bay. Application of the formula, $A_p = 0.255\ A_s$, indicates the potential cross-sectional area of the inlet to be 4,000 to 6,000 sq ft. The original cut was about 1,000 sq ft, which rapidly increased by erosion to over 2,000 sq ft before it was almost completely blocked by a sheet pile cut-off wall installed to stop the erosion. From observation of the eroding pass, it is believed that the cross-sectional area of the pass would assume the 6,000 sq ft, but such a natural pass was not feasible because of limited right-of-way, dense development of adjacent beaches, and proximity of the Intercoastal Waterway.

This potential area formula is, at best, only a rough average approximation, since the pertinent hydraulic dimensions of length, width, and hydraulic radius are not included. It is the first of a series of successive approximations, with refinement added by each step.

DESIGN VELOCITY

The next basic step in designing the size of the inlet introduces length, width, hydraulic radius, and the silting and erosion characteristics of the sand and shell material in the littoral drift. Fundamentally, this procedure simply sizes the inlet to tend toward siltation half the time and toward erosion half the time. The value of the design velocity (V_d) is established at beginning of saltation of the median size material. By trial, the inlet channel is sized until the actual flow velocity at the median tidal differential is the same as the design velocity.

The sampling and testing of the littoral material for size distribution are necessary for use in the formula for this design velocity. The typical, average size distribution of this sand and shell material at four locations is illustrated by Fig. 3. The material at Cedar Bayou and Corpus Christi consists of

[3] "Stability of Coastal Inlets," by Per Bruun and F. Gerritsen, Proceedings, ASCE, Vol. 84, No. WW 3, May, 1958, p. 1644.

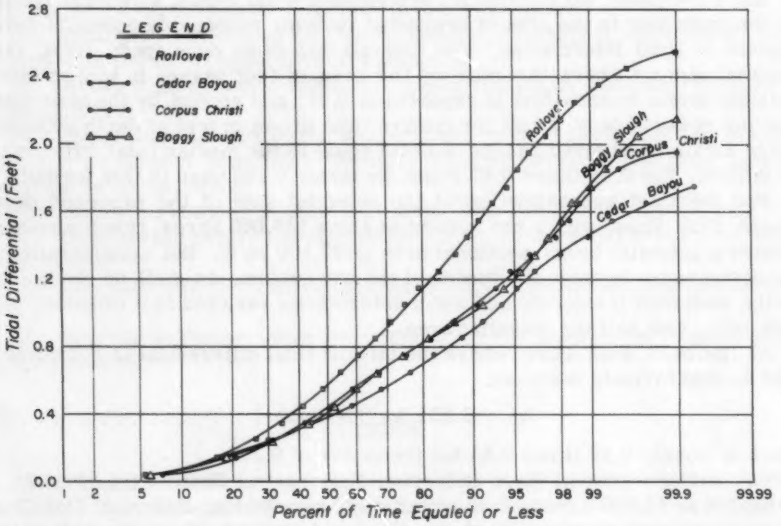

FIG. 2.—TYPICAL FREQUENCY ANALYSES OF TOTAL TIDAL DIFFERENTIAL

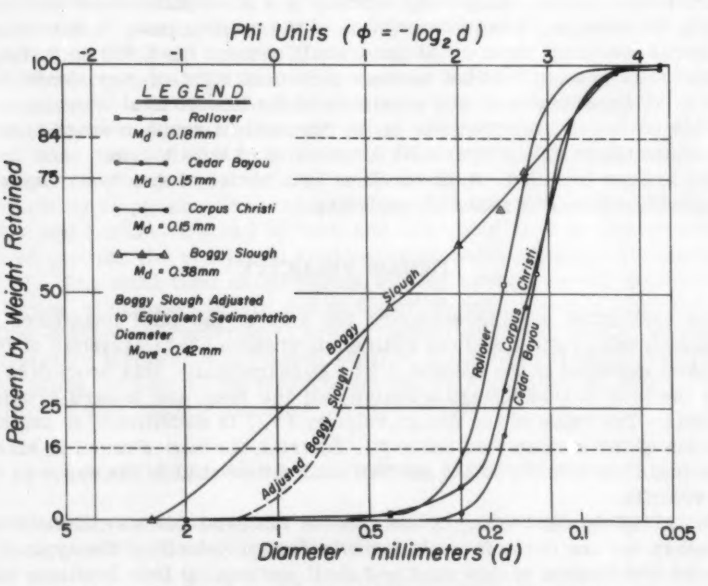

FIG. 3.—SIZE DISTRIBUTION OF LITTORAL MATERIAL

fine, well sorted sand with a median diameter of 0.15 mm. There appears to be no tendency for any further sorting of this beach material, so only the median size is used in the various design analyses.

At Boggy Slough there is extensive distribution of sand and fine shell, with about equal proportions of the two materials. The mean diameter of about 0.77 mm (average of 84% and 16% size) is considerably greater than the median (50% size) diameter of about 0.38. Further study was made of the hydraulic characteristics of the shell, in order to determine whether to use the mean or median in the velocity analyses. The apparent specific gravity of various samples of the shell ranges from 2.70 to 2.86. To correct for the shape and specific gravity of the shell, the fall velocity of various sieve fractions was measured in a large plastic cylinder. This fall velocity was converted to equivalent "sedimentation diameter" by use of a chart of sedimentation diameter versus fall velocity in water.[4]

This conversion materially reduces the hydraulic effective size of the shell graded by sieves above 0.5 mm, as shown in Fig. 3 and noted as "Adjusted Boggy Slough." The adjusted mean diameter is 0.42 mm, as compared with a median diameter of 0.38 mm. Hence, a diameter of 0.40 mm was used in the velocity formula to determine the desired median design velocity for a pass in the shell area.

The desired design velocity is then computed by a formula presented elsewhere.[5] This formula, as slightly simplified by using a specific gravity of 2.65 for quartz sand, is:

$$V_d = \frac{1.486}{n} R^{1/6} (1.65\,k\,d)^{1/2} \qquad \text{. . . . . (3)}$$

in which V_d is the channel velocity in fps for condition of entrainment function (desired design velocity); n refers to the Manning roughness coefficient; R is the hydraulic radius in feet (area divided by wetted perimeter); k is the entrainment function; and d is the diameter of particles in feet (or equivalent sedimentation diameter - with d in mm divided by 304.8 = d in feet).

The entrainment function, k, was secured from a graph.[6]

At the present stage of knowledge, it is considered that the best initial design balance may be secured by providing a pass of such size and length that the actual median velocity due to the tidal differentials is equal to the computed design velocity of saltation (k value of 0.4) for the material encountered in the beach or littoral drift at the site. The significance of the various factors involved in the design velocity is illustrated by the following range of values computed with an average n value of 0.025: for hydraulic radius from 3 to 25, the desired design velocity varies from 1.3 to 1.8 fps for fine sand with median diameter of 0.15 mm, and from 2.1 to 3.0 fps for sand and shell with median diameter of 0.4 mm.

This design velocity (V_d) formula appears to fit reasonably well between the extremes of experience in fish pass behavior, listed in Table 1.

The actual median velocity of various trial sizes and lengths of inlet channel is computed by use of the well known Manning formula, using n values of

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4 "Engineering Hydraulics," Proceedings of the Fourth Hydr. Conf., Iowa Inst. of Hydr. Research, June 12-15, 1949, edited by Hunter Rouse, p. 781.

5 "Water Supply and Waste-Water Disposal," by Gordon M. Fair and John C. Geyer, John Wiley & Sons, Inc., New York, 1954, p. 398.

6 "Engineering Hydraulics," Proceedings of the Fourth Hydr. Conf., Iowa Inst. of Hydr. Research, June 12-15, 1949, edited by Hunter Rouse, p. 790.

about 0.02 to 0.03, for the median tidal differential, with adjustment for losses other than friction. Usually an adjustment of about 1.1 times velocity head will suffice for these other losses. For ready reference, the Manning formula is:

$$V_f = \frac{1.486}{n} R^{2/3} S^{1/2} \quad \ldots\ldots\ldots\ldots\ldots \quad (4)$$

in which V_f refers to the actual flow velocity in feet per second; and S is the slope of water surface or of energy gradient in feet per foot.

After this first trial balance is made, the change in bay tides caused by the pass must be estimated and, where appreciable change is indicated, the tidal differentials are adjusted and used for rebalance.

LITTORAL TRANSPORT

The design procedure cannot go beyond this rebalance without estimates of rates of the total littoral transport from both longshore directions. It is generally considered that the science has not been developed to a degree to insure good quantitative estimates of the littoral transport rates. However, it

TABLE 1.—DESIGN VELOCITY VERSUS MEDIAN FLOW

Pass name (1)	Design velocity V_d, in fps (2)	Median flow V_f, in fps (3)	Characteristic behavior (4)
Rollover	1.6	3.1	Vigorous erosion
Yarborough	2.2	1.0	Rapid closure
Cedar Bayou	1.7	1.3	Tendency to close

does appear that the following procedure for analyses of the wave energy produces reasonable qualitative and quantitative results along coastal Texas.

The longshore component of the usual wave energy must be the major force in the littoral transport, with Gulf current and hurricane waves causing some modification of this usual movement. There are three available wave roses along the Texas coast: off Caplen,[7] Brownsville,[8] and Corpus Christi,[9] which are located respectively at the eastern and southern ends and near the central portion of coast.

The general pattern of the predominant littoral transport is illustrated by Fig. 4. This transport westward at Rollover and Cedar Bayou and northward at Mansfield is well established. The Beach Erosion Board in a report[10] shows this westward drift with beach recession estimated at an annual rate of about 5 ft and deficiency in supply of about 200,000 cu yd annually.

7 "Wave Statistics for the Gulf of Mexico off Caplen, Texas," Tech. Memorandum No. 86 of the Beach Erosion Bd., September, 1956.

8 "Wave Statistics for The Gulf of Mexico off Brownsville, Texas," Tech. Memorandum No. 85 of the Beach Erosion Bd., September, 1956.

9 Unpublished report made to the Gulf Oil Corp. by A. H. Glenn and Assocs.

10 "The Gulf Shore of Bolivar Peninsula, Texas (Erosion at Rollover Fish Pass)," a Beach Erosion Control cooperative study issued April, 1958, by the Galveston Dist. of the Corps of Engrs.

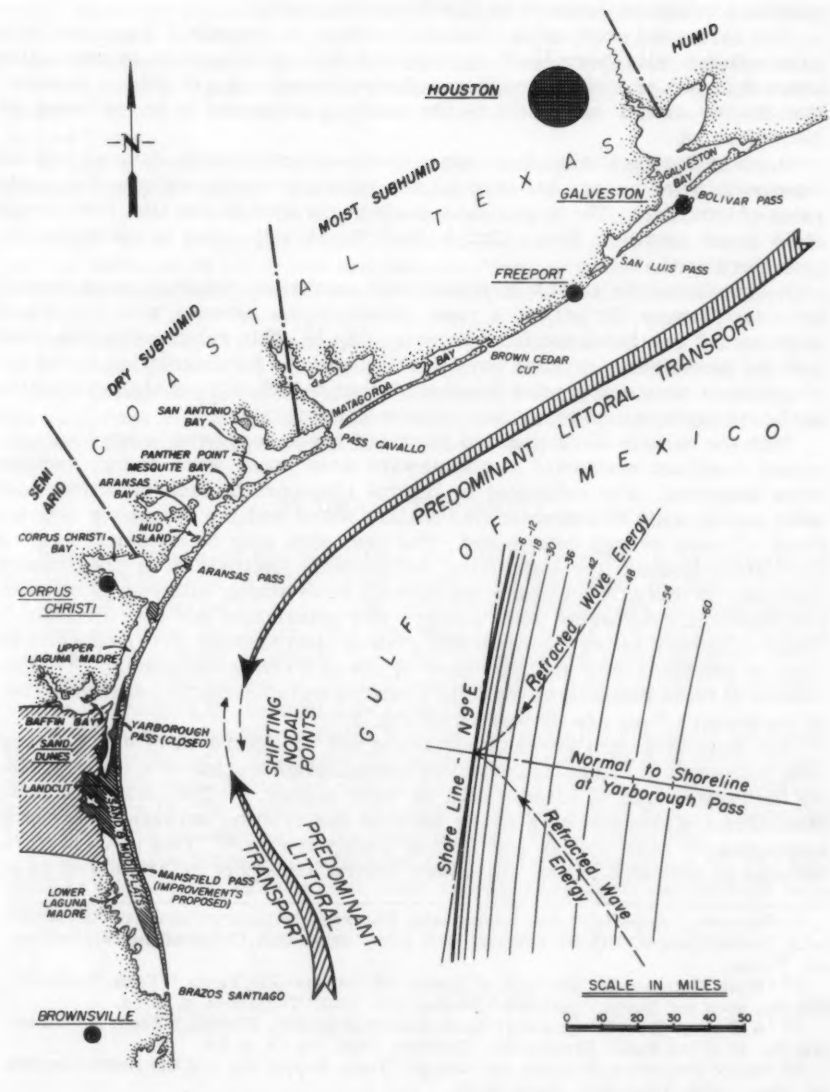

FIG. 4.—WAVE ENERGY AND LITTORAL TRANSPORT

The westward predominant littoral transport at Cedar Bayou is reasonably substantiated by the historic behavior of the Gulf mouth in repeated migrations westward. One estimate of the rate of predominant littoral transport near the beach, made by computed rate of sanding in the updrift Gulf bar as the mouth migrated westward, amounts to 80,000 cu yd annually.

The northward predominant littoral transport at Mansfield Pass has been substantiated elsewhere.[11] It is reported that the shoreline is retreating about 9 ft per yr and, although no definite determination of volume is made, that the net annual movement to the north is estimated to be as much as 300,000 cu yd.

South of Corpus Christi Bay, where there are no available data on littoral transport, wave energy has been used to estimate the character and possible rates of transport. The calculations made at the site of the Old Yarborough Pass (near proposed Boggy Slough Fish Pass) will serve to illustrate the procedure used.

In calculating the annual northward and southward resultant wave energy from the Corpus Christi wave rose, a correlation between wave height and wave period was developed from another table.[12] This relationship was used with the wave energy or total wave work equation,[13] for calculation of the instantaneous wave energy as a function of wave height. Values of the variables in the energy equation were taken from an existing table.[14]

With the various directions and heights of waves converted to wave energy, annual resultant northward and southward deep water wave energy vectors were computed. For estimates of littoral transport in each direction, this wave energy must be refracted into shallow water and the longshore components of wave energy determined. The next step may be considered as an over-simplification in computations, but probably justified for a preliminary analysis. In this step an imaginary solitary wave height, with energy equal to the resultant deep water wave energy, was determined for both directions. These imaginary waves (considerably greater than average wave height) could then be refracted into shallow water by use of a refraction template.[15] The results of these refractions of annual resultant wave energy into shallow water at Yarborough Pass are illustrated on Fig. 4.

The longshore components of northward and southward wave energy were then converted to littoral transport by extrapolation and use of a graph showing the relationship of littoral drift to wave energy.[16] The relation is substantiated and extended by a graph showing the relation between alongshore component of wave energy and littoral transport rate.[17] This gives at Yarborough an indicated annual northward littoral transport of 242,000 cu yd at

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11 "Review of Reports on Gulf Intracoastal Waterway Tributary Channel to Port Mansfield, Texas," issued May 29, 1958, by U.S. Army Engr. Dis., Corps of Engrs., Galveston, Texas.

12 "Wave Statistics for The Gulf of Mexico off Brownsville, Texas," Tech. Memorandum No. 85 of the Beach Erosion Bd., September, 1956, Table B-2, p. B-14.

13 "A Study of Sand Movement at South Lake Worth Inlet, Florida," Tech. Memorandum No. 42 of the Beach Erosion Bd., October, 1953, Eq. 14, p. 18.

14 "Shore Protection Planning and Design," Tech. Report No. 4 of the Beach Erosion Bd., June, 1954, Table D-1, Appendix D.

15 Ibid., Fig. 17, p. 32.

16 "A Study of Sand Movement at South Lake Worth Inlet, Florida," Tech. Memorandum No. 42 of the Beach Erosion Bd., October, 1953, p. 20, and Formula 16, p. 21.

17 "Laboratory Study of The Effect of Groins on The Rate of Littoral Transport," Tech. Memorandum No. 114 of the Beach Erosion Bd., June, 1959, p. 7.

6 ft depth, and 96,000 cu yd southward at 6 ft depth, for a total of 338,000 cu yd.

By similar procedure at Corpus Christi, the estimated annual northward littoral transport is 142,000 cu yd at 6 ft depth, and 112,000 cu yd southward at 6 ft depth, for a total of 254,000 cu yd. Subject to modification by hurricane waves and Gulf currents, it appears that normal wave action shifts the littoral transport almost equally northward and southward at Corpus Christi, but there is a definite net annual northward trend at Yarborough. There should be a greater amount of littoral transport to contend with at Yarborough than at Corpus Christi.

At depths greater than 6 ft, the computed transport rapidly increased to about one-half million cu yd annually, which figure serves to warn of increasing interception of drift with larger passes or longer jetties. An approximate indication of the magnitude of total littoral transport along the Texas coast may be indicated by the annual maintenance dredging at the navigation passes of Brazos Santiago, Aransas Pass, Freeport, and Bolivar, which are estimated respectively at 352,000 - 455,000 - 703,000 and 908,000 cu yd annually.

For the simplified preliminary analyses of littoral transport, the resultant of wave energy on each side of a line normal to the shore line was utilized. For the design stage, calculations will be made on all available segments of the wave rose so that the quantitative value and frequency variation of the estimate of littoral transport rate may be improved.

SEDIMENT TRANSPORT

Whenever the sand and shell from the littoral transport is intercepted or deposited in the inlet, it becomes sediment. The movement of this material, either into the bay or out to the Gulf, by the tidal flow through the pass is designated sediment transport. The next step in design is to rebalance the pass by an operation study to equate the total sediment transport capacity of the inlet with the total amount of littoral transport from both longshore directions that is intercepted by the tidal flow through the inlet.

Of the many and widely variable bed load and sediment transport formulas, the Extended Kalinske Transport Formula is believed to be best suited to this particular purpose. There is a plot of the Kalinske bed-load function presented elsewhere.[18] The formula is extended to include a high rate of suspended transport with movable bed of the same material as in suspension.

The Kalinske formula is presented in several different forms.[18] The formula as adopted and modified for these sediment transport calculations is as follows:

Sediment Transport Function = 10 (Tractive Force Function)^2 or:

$$\frac{q_s}{d\,(g\,R\,S)^{1/2}} = 10\left(0.12\,\frac{\tau}{\tau_c}\right)^2 \qquad (5)$$

or:

$$Q_s = 10\,B\,d\,(1+e)\,(g\,R\,S)^{1/2}\left(0.12\frac{\tau}{\tau_c}\right)^2 \qquad (6)$$

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[18] "Engineering Hydraulics," Proceedings of the Fourth Hydr. Conf., Iowa Inst. of Hydr. Research, June 12-15, 1949, edited by Hunter Rouse, p. 799.

in which q_s is the unit solid volume rate of sediment transport, in cfs per ft width; q refers to the acceleration of gravity = 32.2 fps per sec; R is the hydraulic radius of channel, in ft (area divided by wetted perimeter); S is the slope of the energy gradient (from tidal differentials), in feet per foot; d is the diameter of the sediment particle, in ft, with d in mm divided by 304.8 = d in ft; τ is the tractive force or intensity of bed shear stress, in psf = γ R S, where: γ is specific weight of water or 64 lb per cu ft for salt water; τ_c is the critical tractive force, which is 12 d for quartz sands with specific gravity of 2.65, and is that value of boundary shear at which force of the flow upon the particles just overcomes its resistance to motion; Q_s is the total bulk volume rate of sediment transport in cfs; B is the top width of channel, in ft; and e is the void ratio of sediment, or ratio of volume of voids to volume of solids.

The tractive force function may be computed for quartz sands with a specific gravity of 2.65 when the shell has been adjusted to the equivalent sedimentation diameter of quartz sand. The use of (1 + e) to convert from solid volume rate to bulk volume rate, for comparison with bulk volume estimates of littoral transport, is derived from soil properties depicted elsewhere.[19] A void ratio of about 0.51 may be used for the well sorted fine sand, but field measurements should be made of the void ratio of shell and sand mixtures.

For the transport operation study of a near balanced inlet, the following postualtes are taken:

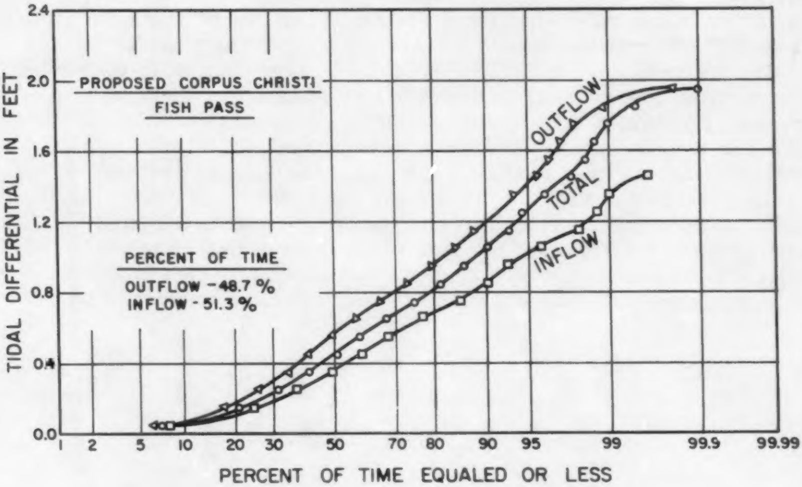
1. When the sediment transport capacity of the tidal flow in the channel is greater than the intercepted littoral transport rate, all intercepted drift material from either longshore direction is carried by inflow to the bay, forming the bay bar, and is carried by outflow to the downdrift side;
2. When the intercepted littoral transport rate exceeds the sediment transport rate of the tidal flow in the channel, the excess drift material is dropped out to form the Gulf bar, during both inflow and outflow.
3. The drift material transported to the bay is utilized to form the bay bar, with some distributed into the bay, mostly by littoral transport, and some (tentatively assumed at about 35%) transported back into the Gulf by outflow.

The application of this formula to the proposed Corpus Christi Fish Pass is illustrated by Fig. 5. This shows the pass estimated to be essentially balanced, with the sediment transport capacity almost equal to the littoral transport out to a depth of 6 ft. The normal sediment transport is first computed without the Gulf bar, with about 32% moved into the bay and 68% moved out to the Gulf.

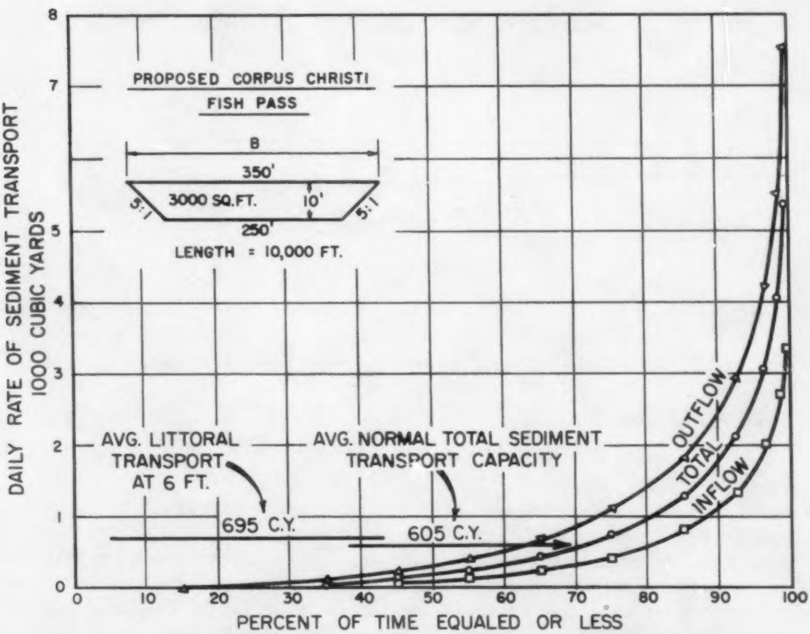
DYNAMIC BALANCE

As the Gulf bar forms within reasonable limits, the velocity of flow is increased over the bar but is decreased through the remainder of the pass. This causes a decrease in the sediment transport through most of the channel, but greatly increases the sediment transport over the bar. Thus, on inflow the bar tends to grow inland, reducing the sediment transport through the normal channel, but on outflow the sediment transport over the bar is considerably

19 "Fundamentals of Soil Mechanics," by Donald W. Taylor, John Wiley & Sons, Inc., New York, 1948, pp. 12 and 13.



(a) INFLOW - OUTFLOW DIFFERENTIALS



(b) SUMMATION OF SEDIMENT TRANSPORT

FIG. 5.—SEDIMENT TRANSPORT

increased. The outward sediment transport increases, as the bar grows, to rates that are greater than the decrease in the inward sediment transport through the normal channel.

The mechanism of the Gulf bar for final dynamic balance is illustrated by Fig. 6. After the nominal sediment transport capacity of the channel is approximately equated to the intercepted littoral transport, the highly variable Gulf bar provides the automatic mechanism to maintain dynamic balance between the various rates of sediment and intercepted littoral transport.

As illustrated on Fig. 5, calculation of the sediment transport rate for the 70% tidal differential will approximate the average rate to indicate total transport. Hence, the effects of the Gulf bar may be estimated approximately by calculation of sediment transport rate with various size bars at this differential. For a definite size and shape of bar, the total sediment transport is

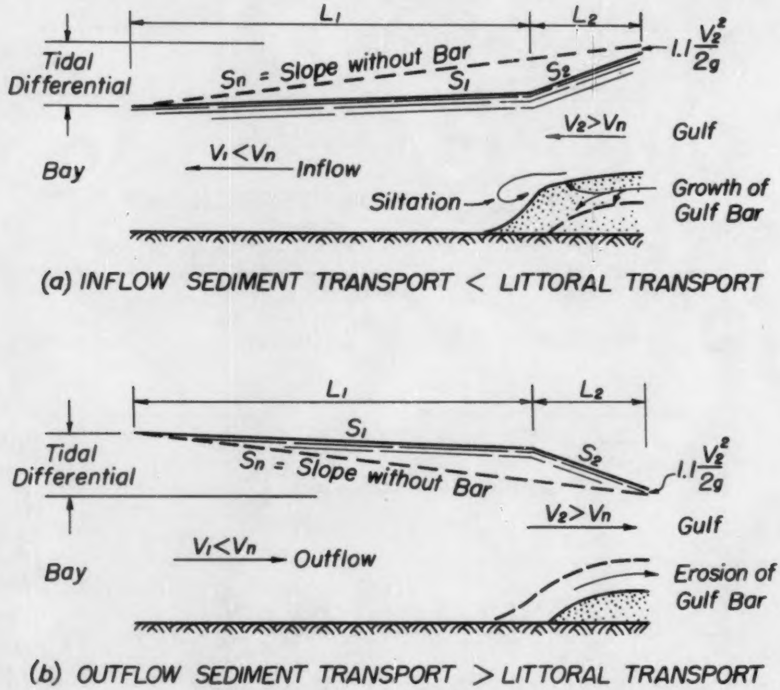


FIG. 6.—MECHANISM FOR DYNAMIC BALANCE

properly computed by increments of the differentials, instead of the more approximate 70% differential.

Two of the several conditions with the changing Gulf bar are illustrated by Fig. 6. When the inflow sediment transport capacity is less than the intercepted littoral transport, the Gulf bar will grow as shown by Fig. 6(a). When the increased outflow sediment transport capacity through the restricted bar section is greater than the intercepted littoral transport, the Gulf bar will erode as shown by Fig. 6(b).

This mechanism may be further illustrated by application to the proposed Corpus Christi Fish Pass. With a Gulf bar that reduces the depth from 10 ft to 8 ft for a length of 500 ft, and the bottom width from 250 ft to 240 ft, the area is reduced 25% and flow about 11%. Velocity is increased over the bar and decreased through the remainder of the pass. The annual sediment transport is increased to 374,000 cu yd (47% greater than annual littoral transport) with only 11% carried by inflow to the bay and 89% carried by outflow to the Gulf. This daily rate is 1,024 cu yd, in comparison with 695 cu yd for the littoral transport.

During short periods of high rates of littoral transport, the depth of water over the Gulf bar will be further decreased. With a Gulf bar that reduces the water depth from 10 ft to 6 ft for a length of 500 ft, and the bottom width from 250 ft to 225 ft, the area is reduced about 48%, with about 22% reduction in flow. The temporary rate of sediment transport is increased to 4,260 cu yd per day, or about 6.1 times the average rate of littoral transport. Most of this sediment transport is accomplished with outflow to the Gulf. Thus, in balanced design, the Gulf bar acts as a natural sand by-passing mechanism with a minimum of sediment transport into the bay.

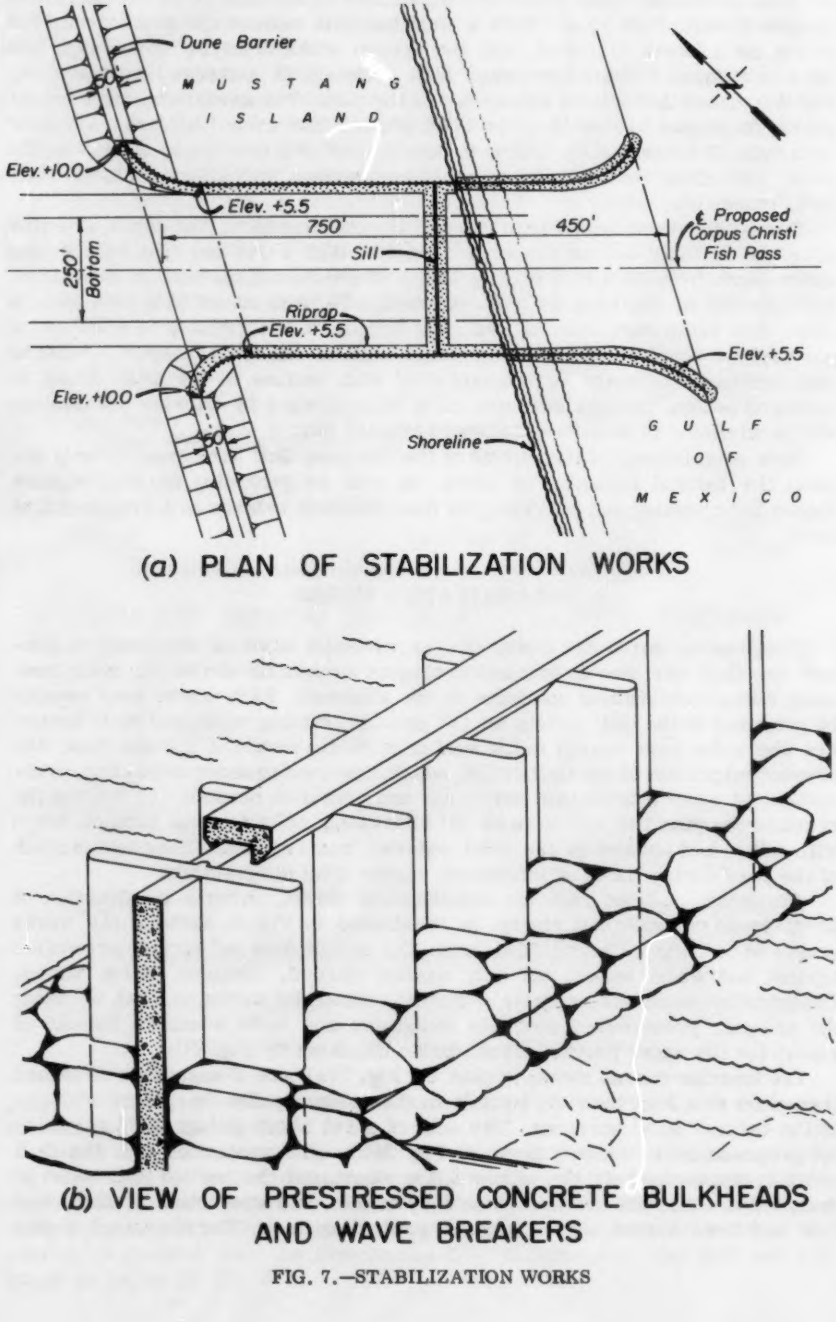
Such calculations of the effects of the changing Gulf bar certainly help explain the natural behavior of inlets, as well as providing an approximate means for checking and providing the final dynamic balance to a proposed fish pass.

STABILIZATION WORKS

Stabilization works are designed on a minimum basis as necessary to control the Gulf bar mechanism and to support imbalance where, for many reasons, completely natural operation is not achieved. Such works may usually be proposed at the Gulf outlets to: (a) provide training walls and wave breakers where the wave energy in the surf must be converted to channel flow; (b) prevent migration of the Gulf outlet, which may tend to move depending on direction of wave attack from day to day and season to season; (c) control the reach of the pass that operates as an alternating siltation and erosion basin with inflow and outflow of the tidal waters; and (d) control the design depth of the pass during times of infrequent, higher tidal differentials.

The latest, untried idea for stabilization works, using a combination of prestressed concrete and riprap, is illustrated by Fig. 7, showing the works proposed at Corpus Christi Fish Pass. The design does not include protection against hurricane waves, but only against normal, frequent storm waves. Comparative economic analyses indicated substantial saving in cost by using the precast, prestressed concrete bulkheads and wave breakers instead of riprap for the upper portion of the works, as shown by Fig. 7(b).

The interior curves shown in plan on Fig. 7(a) are designed as a raised transition to a dune barrier, built from the dredge spoils, to connect with existing natural dune barriers. The sill, of steel sheet piling, will be built to the proposed cross section shown on Fig. 5(b). The projection into the Gulf is up to approximately the minus 6 ft contour, and the curved Gulf outlet is designed to eliminate the undesirable hydraulic characteristics of disturbed flow and head losses with a straight, projecting inlet. The alignment of this



(a) PLAN OF STABILIZATION WORKS

(b) VIEW OF PRESTRESSED CONCRETE BULKHEADS AND WAVE BREAKERS

FIG. 7.—STABILIZATION WORKS

proposed Corpus Christi Fish Pass is S 45° E, conforming with the prevailing wind to minimize the invasion of the pass by shifting wind-blown sands, as well as to improve water interchange through the pass.

SALINITY CONTROL

The average salinity of the Gulf water is about 35 parts per thousand (ppt). The salinity of the bay waters varies from about 1 ppt to 100 ppt. Subject to much additional research and substantiation by the marine biologist, it is tentatively assumed that, where feasible, it will be desirable to control the salinity of the bay waters within the general range of about 20 ppt to 50 ppt.

Salinity currents, or density currents in the form of stratified flow or gravity underflows, must occur in the bays as a result of the horizontal and vertical gradients in salinity, which are a direct measure of density differences. Horizontal salinity gradients are illustrated by Fig. 9, showing profiles of salinity in East Galveston Bay before Rollover was cut and later while it was rapidly enlarging by erosion. Data for the before profile were developed from a study presented elsewhere.[20] Data for the after profile were secured from another publication.[21]

Vertical salinity gradients have been measured often by the marine biologist in the various bays at about 0.5 ppt to 3 ppt from top to bottom, with occasional vertical differentials of 6 ppt to 14 ppt. Horizontal salinity gradients of 10 ppt in 5 miles to 20 miles were often measured, and sometimes gradients as steep as 10 ppt in 1.8 miles. These figures come from both unpublished data and published articles.[22,23]

The following modification of the normal Manning hydraulic equation for turbulent flow has been developed to evaluate the probable effects of salinity currents on the present and future distribution of salinities and on water inchange:[24]

$$V_{st} = \frac{1.486}{n} R^{2/3}\left(\frac{D\ \Delta\gamma\ \Delta\gamma h}{L\gamma}\right)^{1/2} \qquad \text{. . . . . (7)}$$

in which V_{st} is the velocity of salinity current with turbulent flow, in fps; n denotes the Manning roughness coefficient; R refers to the hydraulic radius, in feet, and here taken as one-half the depth of underflow, in order to include the interface between the stratified flows in the length of wetted perimeter.[25] D is the depth of water, in feet; $\Delta\gamma$ is the difference between density of the water at top and bottom (from vertical salinity gradients); γ is the density of the water at bottom; $\Delta\gamma_h$ refers to the difference between the average up-

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20 "A Summer Study of the Biology and Ecology of East Bay, Texas," by George K. Reid, Jr., Texas Journal of Science, Vol. VII, No. 3, September, 1955.

21 "Biologic and Hydrographic Adjustment in a Distrubed Gulf Coast Estuary," by George K. Reid, Limnology and Oceanography, Vol. II, No. 3, July, 1957.

22 "An Introduction to the Hydrography of Tidal Waters of Texas," by Albert Collier and Joel W. Hedgpeth, publications of the Inst. of Marine Science, Vol. I, No. 2, November, 1950, pp. 125-194.

23 "An Ecological Survey of the Upper Laguna Madre of Texas," by Ernest G. Simmons, publication of the Inst. of Marine Science, Vol. IV, No. 2, July, 1957, pp. 156-200.

24 "Thermal Density Underflow Diversion, Kingston Steam Plant," by Elder & Dougherty; and "Submerged Sluice Control of Stratified Flow," by Harleman, Gooch and Ippen, Proceedings, ASCE, Vol. 84, No. HY 2, Part 1, April, 1958.

25 Ibid., p. 1583-9.

stream density and average downstream density of the water (from horizontal salinity gradients); and L is the length of reach, in feet.

It has been stated that a number of authorities have shown that stratified flow calculations can be made by using normal hydraulic equations if the gravitational term for the underflow is multiplied by the ratio of the difference in density between the underflow and the overlying layer to the density of the underflow.[25] In that paper the factor or ratio was applied to the Chezy equation. In this paper the factor is applied to the normal Manning velocity equation. The delta gamma divided by gamma, in the portion raised to the one-half power, is this gravitational adjustment.

The remainder of the portion raised to the one-half power is derived from the fact that differences in horizontal salinity cause differences of density which cause differential pressure. The general principle has been stated that the pressure force tends to make water flow from a region of high pressure toward a region of low pressure.[26] In uniform depth of water with level water surface, the energy gradient must be due to the difference in weight of the upstream and downstream columns of water. This difference in weight may be converted to equivalent height or head of water and, when divided by length of reach, will represent the slope of the energy gradient, S. The depth times delta gamma sub-h divided by length, in the portion raised to the one-half power, represents the slope of the energy or pressure gradient in feet per foot. This is herein termed pure salinity current under turbulent flow, and is illustrated by Fig. 8(a).

The hydraulic radius of the underflow is equal to one-half the depth of underflow. The hydraulic radius of the overflow is equal to the depth of overflow. Theoretically, differences in the coefficient of roughness, n, between the bottom and interface cause the relative depths of underflow and overflow to vary. For instance, with equal n values, the depth of underflow is estimated at 0.57 D. For an n value of 0.01 at the interface and a value of 0.03 at the bottom, the depth of underflow is estimated at 0.67 D.

For shallow bays with low vertical and horizontal salinity gradients, the Reynolds Number of the estimated turbulent flow falls below the lower critical Reynolds Number of 2,000, so a formula for laminar flow of the salinity current is needed for complete analyses. Such a formula has been derived from a formula presented elsewhere.[27]

$$V_{sl} = \frac{20.8\ D\ \Delta\gamma\ \Delta\gamma_h\ R^2}{L\ \mu} \quad \ldots\ldots\ldots\ldots\ldots \quad (8)$$

in which V_{sl} is the velocity of pure salinity current with laminar flow, in fps; D refers to the total depth of water, in feet; $\Delta\gamma$ is the difference in density at top and bottom, from vertical salinity gradients; $\Delta\gamma_h$ designates the difference in density at upstream and downstream limits of reach from horizontal salinity gradients; R is the hydraulic radius of the underflow, in feet, equal to one-half the depth of underflow; L is the length of reach, in feet; and μ is the dynamic viscosity of the underflow, in lb-sec per sq ft, about 2.79×10^{-5} for sea water, but varies with temperature, salinity, and turbidity.

The basic formula is modified by making the fundamental gravitational adjustment of difference in vertical density divided by bottom density, and use

26 "Gulf of Mexico, Its Origin, Waters and Marine Life," Fishery Bulletin 89, Vol. 55, of the Fish and Wildlife Service, U. S. Printing Office, 1945, p. 122.

27 "Engineering Hydraulics," Proceedings of the Fourth Hydr. Conf., Iowa Inst. of Hydr. Research, June 12-15, 1949, edited by Hunter Rouse, Formula 82, p. 79.

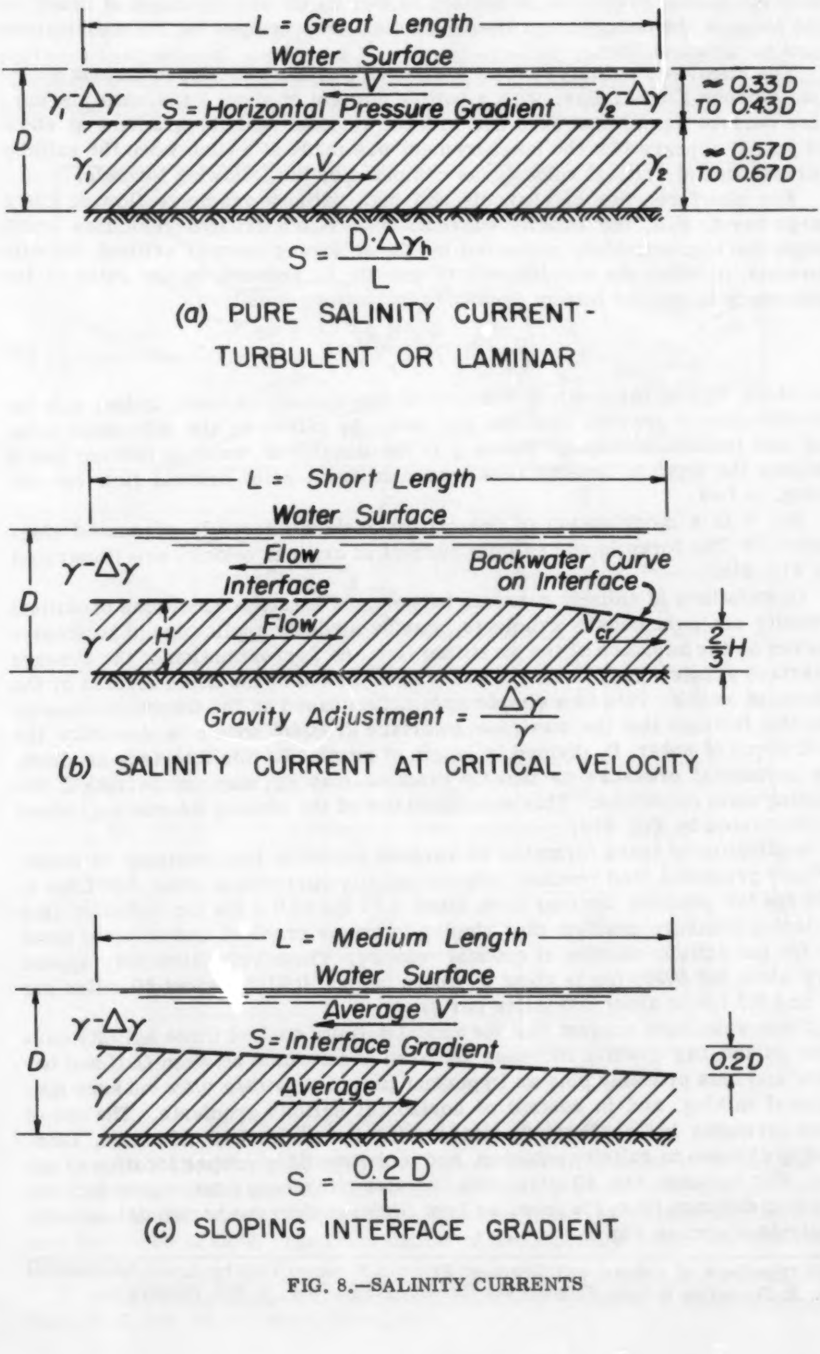

FIG. 8.—SALINITY CURRENTS

of depth times difference in horizontal density divided by length of reach for the slope of the energy or pressure gradient as developed for the salinity current in turbulent flow.

The maximum salinity current measured was from the Upper Laguna Madre into Corpus Christi Bay, with a bottom current of about 1 fps while the surface current was almost zero and with the vertical salinity gradient at about 10 ppt. It appears that the measurement was made at a time when the salinity current was at critical velocity, as computed by the following formula.

For short reaches of channels and high salinity gradients flowing into a large bay or gulf, the salinity currents may reach critical velocities which might be approximately computed by the following normal critical velocity formula, in which the acceleration of gravity is reduced by the ratio of the difference in top and bottom density to the bottom density:

$$V_{cr} = \left(\frac{2\,g\,H\,\Delta\gamma}{3\,\gamma}\right)^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

in which V_{cr} is the critical velocity of the salinity current, in fps; g is the acceleration of gravity, 32.2 fps per sec; $\Delta\gamma$ refers to the difference in the top and bottom density of water; γ is the density of water at bottom; and H denotes the depth of density flow upstream from point critical flow can develop, in feet.

Eq. 9 is a modification of the critical velocity formula presented elsewhere.[28] The formula and salinity current at critical velocity are illustrated by Fig. 8(b).

Calculations of salinity currents between the extreme conditions of critical velocity and pure density gradients, may be made by application of backwater curves on the interface of the stratified flow. An approximation of the average interface gradient may be on the order of about 0.2 of the depth divided by the length of reach. This is a simple assumption based on the theoretical analyses that indicate that the maximum interface gradient should be one-third the total depth of water, D, divided by length of reach. To this interface gradient, the horizontal pressure or density gradient may or may not be added, depending upon conditions. This simplified use of the sloping interface gradient is illustrated by Fig. 8(c).

Application of these formulas to various probable combinations of depth, salinity gradients, and reaches indicate salinity currents of about 0.001 fps to 0.01 fps for possibly laminar flow, about 0.01 fps to 0.1 fps for turbulent flow including pressure gradient plus sloping interface gradient, and on up to about 1.0 fps for salinity current at critical velocity. These velocities may appear very slow, but 0.005 fps is about 30 miles per yr, 0.01 fps about 60 miles per yr, and 0.1 fps is about 600 miles per yr.

These velocities suggest that for annual salinity control these salinity currents in the bay greatly increase the area of interface between Gulf and bay water and thus probably play an important part in providing slow but sure mechanical mixing, and in control of horizontal salinity gradients. The use of these formulas is beneficial in interpreting measured bay salinities, forecasting changes in salinity patterns, and in determining proper location of inlets. For instance, the effectiveness of water interchange decreases with increasing distance from the inlet, as best illustrated by the horizontal salinity gradients shown on Fig. 9.

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28 "Handbook of Culvert and Drainage Practice," issued 1950 by Armco and printed by R. R. Donnelley & Sons, Chicago and Crawfordsville, Ind., p. 231, Chapter 5.

There is considerable evidence that the Coriolis force (a deflecting force due to the earth's rotation diverting horizontal motions to the right in the northern hemisphere) must have some influence on the tidal inflow-outflow, on the distribution and direction of horizontal salinities and salinity currents and, hence, on the efficiency of mechanical mixing. For instance, in Fig. 9,

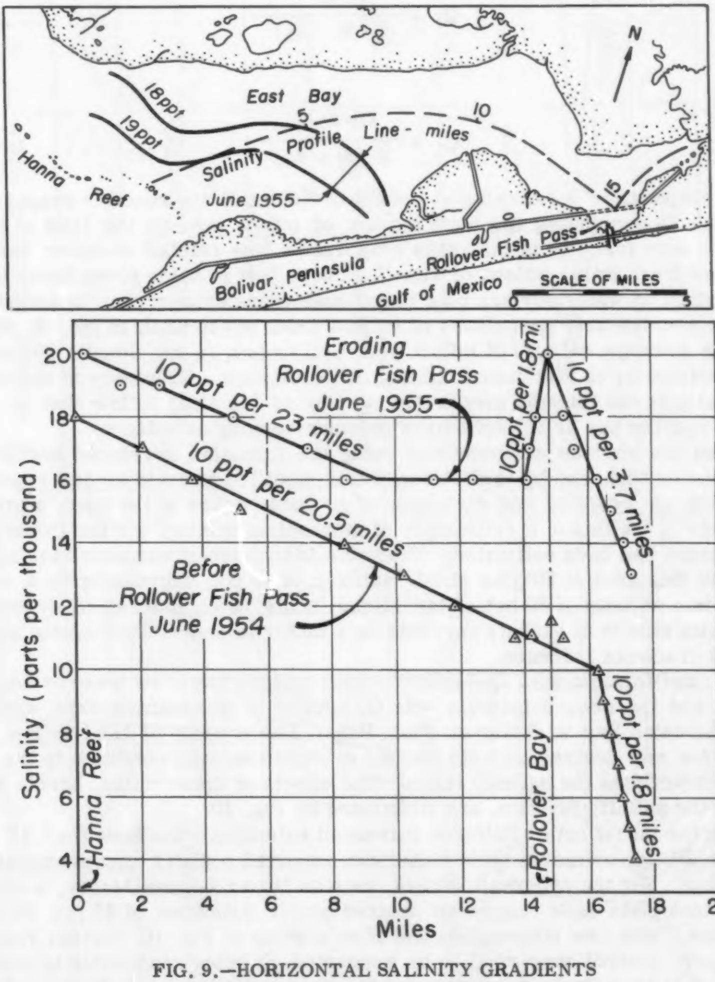

FIG. 9.—HORIZONTAL SALINITY GRADIENTS

the shapes of the 18 and 19 ppt isohalines in East Bay, June, 1955, suggest effects of the Coriolis force. The slight deflecting force becomes materially effective in the wide portions of the bays and in larger passes with high volume and rates of flow. The formulation of the Coriolis force has been well illustrated.[29]

[29] "Rivers Under Influence of Terrestrial Rotation," by Otakar W. Kabelac, Proceedings, ASCE, Vol. 83, No. WW 1, April, 1957.

The basic formulation for inlet inflow and outflow under stationary salinity conditions is presented elsewhere.[30] Since the basic concept does not include the important requirement for mechanical mixing in estimating tidal water interchange for given stable salinity conditions, the formulas has been modified and extended as follows:

$$Q_i = \frac{Q_e\, S_o}{E\,(S_o - S_i)} \quad \text{.............. (10a)}$$

or

$$Q_i = \frac{Q_r\, S_o}{E\,(S_i - S_o)} \quad \text{.............. (10b)}$$

for: (evaporation > rainfall and runoff) or (rainfall and runoff > evaporation in which Q_i designates the total amount of inflow through the inlet in given time, in acre feet; Q_e is the gross evaporation less rainfall on water surface and less fresh water inflow, or runoff, in acre feet in same given time; Q_r is the rainfall on water surface plus runoff less gross evaporation, in acre feet; S_o refers to the average salinity of outflow from bay to Gulf, in ppt; S_i signifies the average salinity of inflow from Gulf to bay, in ppt, usually 35; and E is the efficiency of mechanical mixing, in percentage. Efficiency of mechanical mixing in the formula means that portion of the tidal inflow that is well mixed with the bay or lagoon waters before returning as outflow.

From the various computations using the formulas developed herein for salinity currents, and by application of this modified formula to the reported salinities as weighted and estimates of probable inflow at the time, a probable range of the factor E (efficiency of mechanical mixing) for the Upper Laguna Madre has been estimated. The value is tentatively estimated for application to this area at 20% for stable salinity of 40 ppt, increasing to a value of 40% at a salinity of 80 ppt. This sliding scale is considered to be due to increasing effects of salinity currents as a factor in mechanical mixing as the salinity gradients increase.

The first formula with Q_e applies to high salinity bays, such as the Laguna Madre, and the second formula with Q_r applies to low salinity bays, such as East Galveston Bay at Rollover Fish Pass. The amount of tidal inflow required for any desired salinity control or stable salinity condition in the bay varies directly as the salinity ratios. The effects of these ratios, herein designated the salinity function, are illustrated by Fig. 10.

When the initial cut at Rollover increased salinities from less than 12 ppt to about 20 ppt or more, sport fishermen reported positive improvements in the fishing. For the hypersaline bays, such as Upper Laguna Madre, various marine biologists have suggested desired stable salinities of 45 ppt, 50 ppt, and 60 ppt. With this information and after a study of Fig. 10, certain ranges for salinity control may readily be suggested as being reasonable to attain. That is, it is tentatively suggested that the design limits for salinity control be about 17.5 ppt to 25 ppt in the low salinity bays and about 47.5 ppt to 55 ppt in the hypersaline bays. The design for annual salinity control is made for wet years in the humid climate with low salinity bays, and for dry years in the semi-arid climate with hypersaline bays.

30 "The Oceans - Their Physics, Chemistry and General Biology," by Sverdrup, Johnson and Fleming, 4th Printing, Prentice-Hall, Inc., New York, 1952, p. 147.

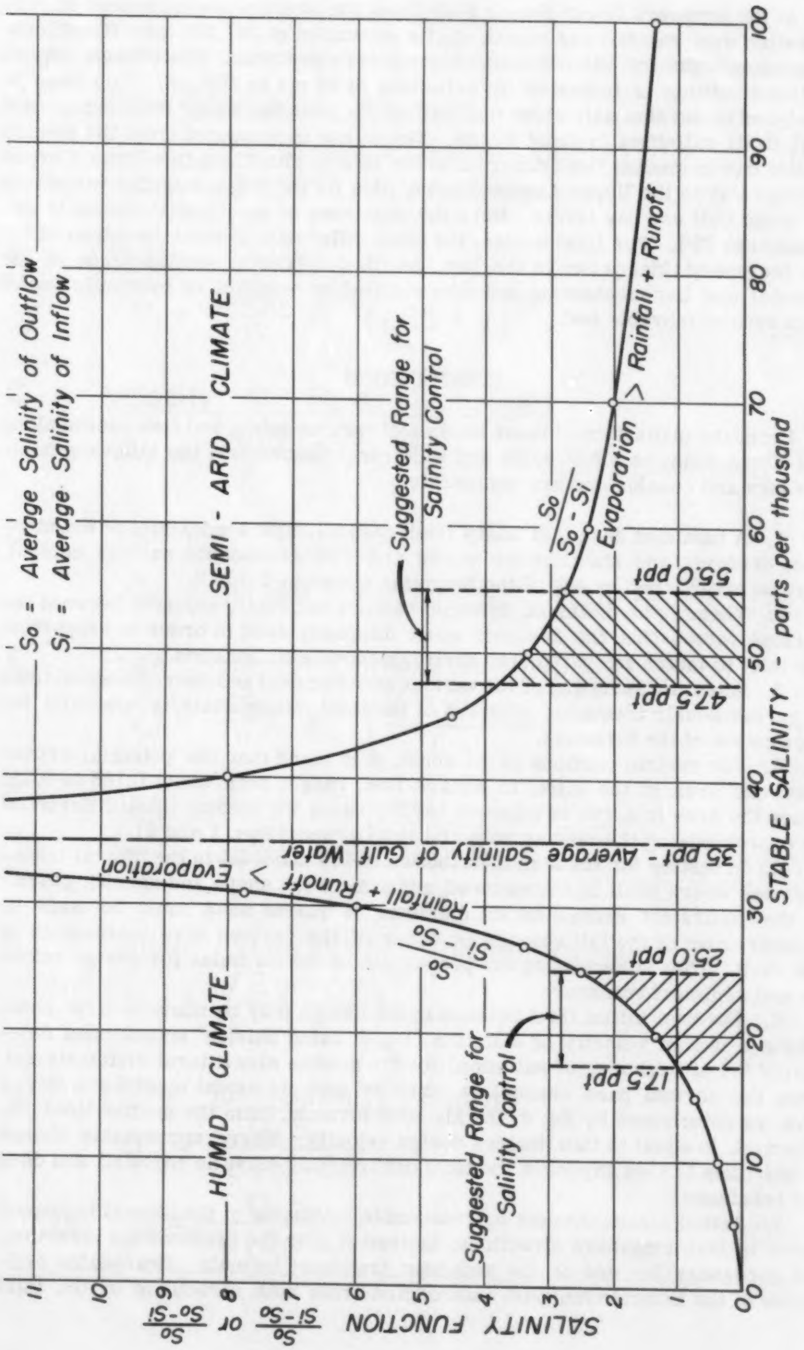

FIG. 10.—SALINITY CONTROL

At the proposed Boggy Slough Fish Pass, the average annual excess of evaporation over rainfall and runoff, Q_e, is estimated at 347,500 acre ft, with the maximum value of 861,000 acre ft for a very dry year. The present hypersaline condition is indicated by salinities of 85 ppt to 100 ppt. This pass is designed to develop only about one-half of the potential water interchange, and will limit salinities to about 50 ppt. The inflow is computed from the median inflow due to median tidal differential for inflow, plus the inflow from Corpus Christi Bay to the Upper Laguna Madre, plus inflow due to monthly variations in mean Gulf and bay levels. Here the efficiency of mechanical mixing is estimated at 25%. For final design, the tidal differentials must be adjusted for the increased tidal prism in the bay, detailed operation studies made of the channel and bar mechanism and then verified or modified by hydraulic model with special movable bed.

CONCLUSIONS

From the utilization of these studies of various inlets and fish passes along the Texas coast as a full-scale and wide range laboratory, the following basic findings and conclusions are enumerated:

1. A balanced design of inlets (fish passes), with a minimum of maintenance dredging and stabilization works and with reasonable salinity control, may be approached by use of the formulas shown in Table 2.
2. There is an optimum, dynamic balance delicately adjusted between the various forces that the designer must diligently seek in order to proportion the inlet to match the particular environment at each location.
3. Scientific sampling of the various astronomical and meteorological tides to permit hourly frequency analyses of the tidal differentials is essential for application of the formulas.
4. For certain portions of the coast, it is found that the potential cross-sectional area of the inlet, in square feet, ranges from about 0.185 to 0.255 times the area in acres of adjacent bay, by using the median tidal differential as approximating the median potential tidal prism (Eqs. 1 and 2).
5. In testing for the size distribution of the material in the littoral transport and where shell is encountered, adjustment for shape and specific gravity to the equivalent sedimentation diameter of quartz sand must be made by measurement of the fall velocity in water of the various size increments of the shell. This is necessary for proper use of the formulas for design velocity and sediment transport.
6. The best initial trial balance in the design may be made by first computing a design velocity of saltation (Eq. 3, using Shields' entrainment function of 0.4 at beginning of saltation) for the median size littoral drift material. Then the normal pass channel is sized so that its actual medial velocity of flow, as determined by Eq. 4, the Manning formula, from the median tidal differential, is equal to this desired design velocity. Where appreciable change in bay tides is then expected, the tidal differentials must be adjusted, and used for rebalance.
7. Actual measurements or reasonable estimates of the littoral transport rates in both longshore directions, instead of only the predominant direction, are necessary for use of the sediment transport formula. Reasonable estimates of the littoral transport intercepted from both directions by the inlet

TABLE 2.—SUMMARY OF FORMULATION

A Potential Area of Inlet

$$A_p = 0.185\, A_s \quad \text{(Central Coast)} \qquad (1)$$

$$A_p = 0.255\, A_s \quad \text{(Rollover)} \qquad (2)$$

B Design Velocity

$$V_d = \frac{1.486}{n} R^{1/6} (1.65\, k d)^{1/2} \qquad (3)$$

C Velocity of Flow (Manning)

$$V_f = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad (4)$$

D Sediment Transport

$$Q_s = 10\, B d (1 + e)(g R S)^{1/2} \left(0.12 \frac{\tau}{\tau_c}\right)^2 \qquad (6)$$

E Pure Salinity Current - Turbulent

$$V_{st} = \frac{1.486}{n} R^{2/3} \left(\frac{D \cdot \Delta\gamma \cdot \Delta\gamma_h}{L \cdot \gamma}\right)^{1/2} \qquad (7)$$

F Pure Salinity Current - Laminar

$$V_{sl} = \frac{20.8\, D \cdot \Delta\gamma \cdot \Delta\gamma_h \cdot R^2}{L \cdot \mu} \qquad (8)$$

G Critical Velocity of Salinity Current

$$V_{cr} = \left(\frac{2 g H \cdot \Delta\gamma}{3 \cdot \gamma}\right)^{1/2} \qquad (9)$$

H Inflow for Salinity Control

$$Q_i = \frac{Q_e S_o}{E (S_o - S_i)} \qquad (10a)$$

or

$$Q_i = \frac{Q_r S_o}{E (S_i - S_o)} \qquad (10b)$$

may be made along this coast from the computed longshore components of the normal wave energy.

8. Rebalance of the inlet in length and area may then be made by use of the Kalinske sediment transport and tractive force functions, Eq. 6, to equate the total sediment transport capacity of the inlet channel to the intercepted littoral transport.

9. Final dynamic balance may then be secured by use of the highly variable Gulf bar as a silting basin when the sediment transport capacity of the inlet is less than the intercepted littoral transport rate and as an erosion basin when the sediment transport capacity of the inlet is greater than the intercepted littoral transport rate. The design intent is to have this variable Gulf bar act as a natural sand by-passing mechanism with a minimum of sediment transport into the bay.

10. Stabilization works should be designed on a minimum basis and placed only as necessary to control the Gulf bar mechanism and to support imbalance where complete natural operation may not be achieved.

11. The most convenient measure of water interchange is in terms of salinity control. It is found that the most feasible annual salinity control in the bays ranges from about 17.5 ppt to 25 ppt in the humid climate where rainfall and runoff exceeds evaporation and about 47.5 ppt to 55 ppt in the semi-arid climate where evaporation exceeds rainfall and runoff (Eq. 10a and b). It is believed that the marine biologist will find that these feasible ranges also approximate the salinity ranges desired from the fisheries standpoint.

12. With each cycle of tidal inflow and outflow only a variable portion of the Gulf water becomes completely mixed with the bay water to effect salinity control, termed efficiency of mechanical mixing. The evidence of vertical and horizontal salinity gradients indicate that salinity or density currents, ranging from possibly laminar through turbulent to critical flow, materially affect this efficiency (Eqs. 7, 8, and 9).

13. To verify and refine the design there is now a distinct need for the use of hydraulic models with movable bed to complete the design stage and then the construction of a fish pass balanced in accordance with criteria and formulas developed herein. As is usual with such a design in its infancy, many refinements will be developed when such fish passes are constructed and after thorough scientific checks are made of their behavior.

ACKNOWLEDGMENTS

Much of the foregoing material was included in an engineering report, titled "Development Report on Fish Passes and Water Interchange for The Upper Laguna Madre and Corpus Christi Bay, Coastal Texas" and "Sediment Transport Supplement," prepared for the Game and Fish Commission, State of Texas, by the firm of Lockwood, Andrews & Newman. The Commission has consented to publication in whole, or in part, of the engineering information gathered while the firm was employed as consultant on fish passes. H. D. Dodgen, Executive Secretary of the Commission, has expressed the hope that by this publication others will be able to avoid some pitfalls as a result of the experiences reported. Conversely, the anticipated comments and discussions on this paper will add useful information to the subject.

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

SOUTHWEST PASS-MISSISSIPPI RIVER 40-FT CHANNEL

By Austin B. Smith,[1] F. ASCE

SYNOPSIS

The paper examines the heavy high water shoaling, triggered by action of the salt-water wedge, and outlines plans to obtain the authorized 40-ft deep channel connecting the Mississippi River and the Gulf via the Southwest Pass jettied channel and across the Sea Bar entrance channel. The magnitude of the annual shoaling and the associated salt-water intrusion phenomenon, prior jetty and pile dike construction, current hopper dredge requirements to maintain 35-ft-deep ship channel, the collection of new field data, and prototype studies and model tests of various improvement schemes are described. The formulated plan for providing the authorized 40-ft-deep ship channel and reducing the burdensome annual hopper dredging requirements is presented.

GENERAL DESCRIPTION

The Corps of Engineers is improving Southwest Pass to provide a dependable 40-ft-deep ship channel connecting the main stem of the Mississippi River with the Gulf of Mexico. The deep water navigation project, Mississippi River, Baton Rouge to the Gulf of Mexico, is shown in Fig. 1. The central problem is the elimination of heavy high water shoaling that forms near the mouth of Southwest Pass during each high water season. To find the best solution to this 50-yr-old problem, prototype studies and investigations and model tests of a number of possible schemes of improvement were made. These studies and tests indicated that a more sinuous channel alignment below mile 9 below head of

Note.—Discussion open until February 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 3, September, 1960.

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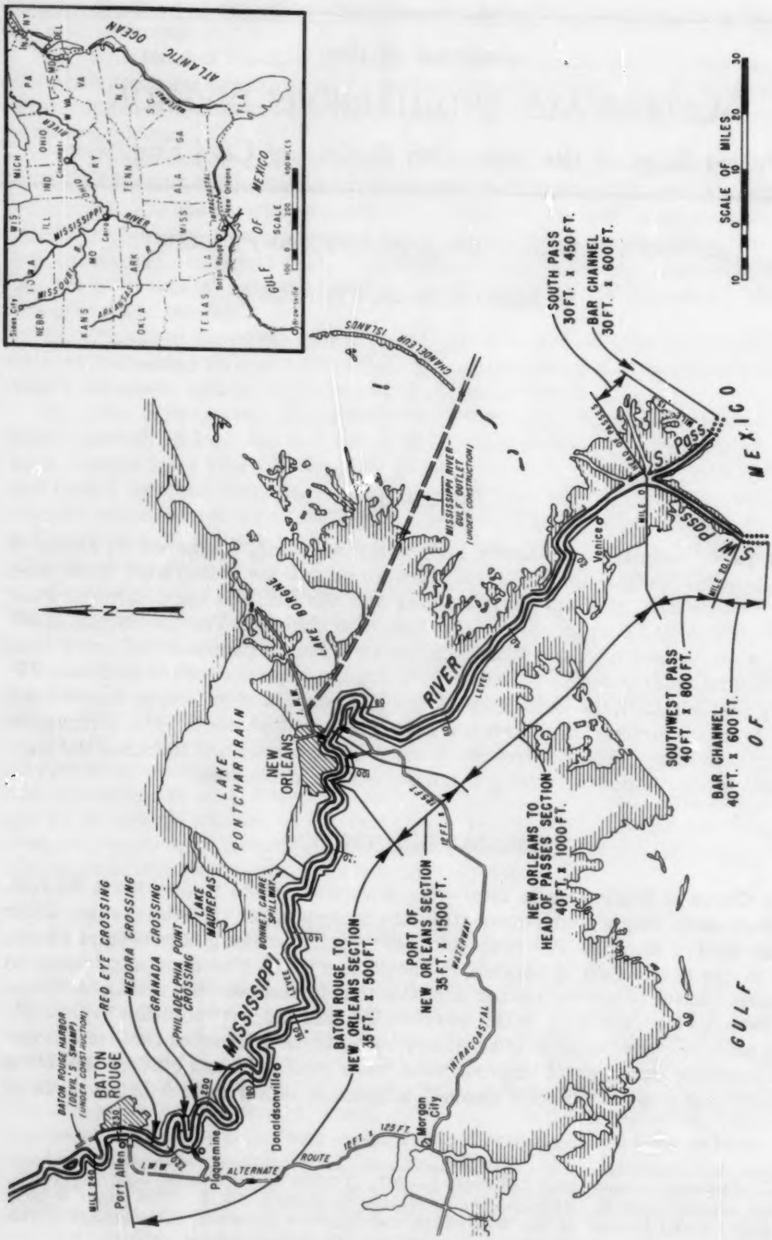

FIG. 1.—MISSISSIPPI RIVER, BATON ROUGE TO THE GULF OF MEXICO, NAVIGATION PROJECT

passes (BHP), and particularly in the jettied sector below Mile 17 BHP, will give additional force to the high, fresh-water flow.

MISSISSIPPI RIVER DELTA

The Mississippi River brings to its mouth sediment loads in the order of one million tons daily.[2] These land-building materials are moved mainly during the annual high water between January and July. The overlapping delta now reaches within 10 mi of the landward edge of the continental slope. Early explorers and mapmakers appropriately called the Mississippi River Delta the "Cabo de Lodo" or "Mud Cape."[3] The problem area, shown in Fig. 2, lies at the seaward end of the delta where the Mississippi River flow via Southwest Pass meets the Gulf.

Landmarks bear witness to the Mississippi River Delta's advance. The French erected the town of Old Balize at the mouth of the Northeast Pass (Pass a Loutre) in 1774. In 1834 it stood 2 mi above the mouth. In 1858 the rate of advance of new land was believed to be 96 ft a year.[4] Fig. 3 shows the seaward advance of Southwest Pass Sea Bar since the 1898 survey. The average annual advance of the 35-ft contour since 1898 has been approximately 100 ft. This rate of advance should decrease as the delta advances into deeper waters of the Gulf.

HISTORIC NAVIGATION PROBLEM

In the 400 years since European explorers discovered the mouth of the Mississippi, engineers of several nationalities have been concerned with the problem of improving the Mississippi River Passes for navigation. French, Spanish, and British engineers worked at the problem from circa 1700 to 1812. Their legacies include maps of early surveys, Bar Pilots, light houses, and river towns that are now landmarks. They named the passes for the direction in which they run.

The United States began to survey and report on the problems of deep water navigation through the passes in 1829. Countless engineers have since been concerned with this much storied problem. The records are highlighted by such names as Beauregard, Humphreys, Eads, Corthell, Ockerson, Schultz, Lipsey, Dent and Townsend, and more recently by Tyler, Ferguson, Hudson, Tiffany, Cobb, Gentilich, Kennedy and Simmons.

The problem in the 17th and 18th centuries was to find 12 to 15 ft for ships via Pass a Loutre. In the 19th century, it was to provide a 30-ft ship channel via South Pass. In the 20th century the problem was first to obtain 35-ft and now 40-ft through Southwest Pass.

The first Federal appropriation was made in 1836 and 1837 to improve Pass a Loutre. The chronicle of feeble attempts to improve the passes continued through 1875. The controversy that raged from 1871 to 1875 over whether to improve the channel of one of the passes channels by means of a jetty system or to provide a connecting channel through a canal and lock was settled in 1875

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[2] "Sedimentary Framework of the Modern Mississippi Delta," by H. N. Fisk, et al, June, 1954.

[3] "Gulf of Mexico, its Origin, Waters, and Marine Life," U.S. Department of Interior, 1954.

[4] "Report on Physics of the Mississippi River," by Humphreys and Abbott, 1861.

when Congress accepted Eads' "No cure, No pay" proposition. This proposition was "to secure by construction of jetties and auxiliary works and maintain for 20 years a channel 26 ft deep by 200 ft wide having through it a central depth of 30 ft without regard to width," for the sum of $8,000,000. Eads de-

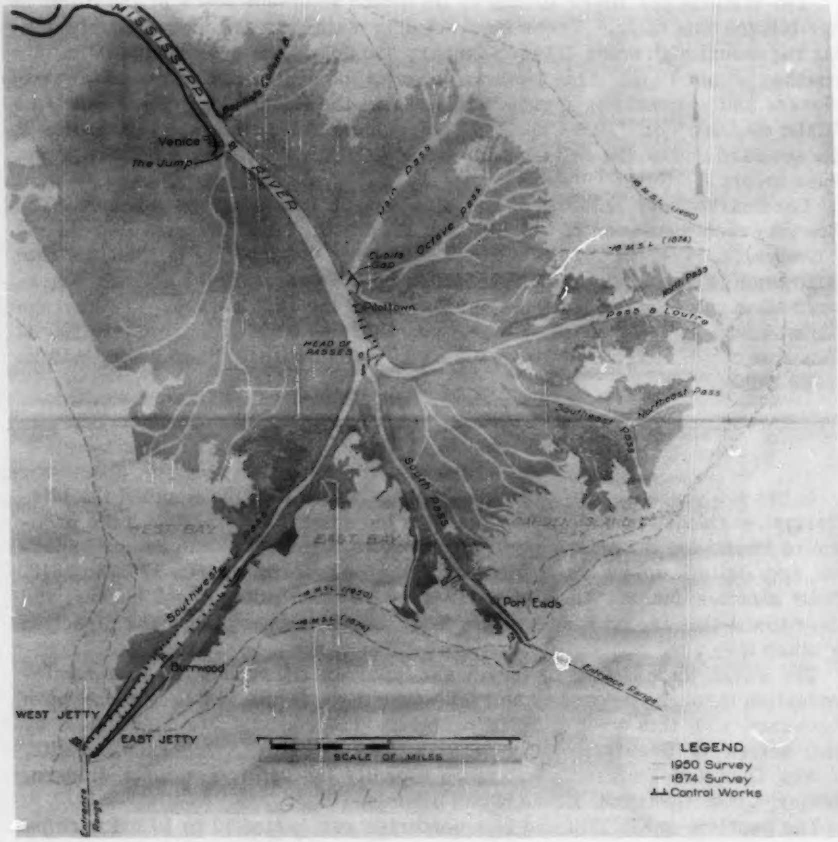

FIG. 2.—DELTA AND PASSES, MISSISSIPPI RIVER

sired to improve Southwest Pass, but the Congress directed that he experiment with South Pass.[5] The impact of Eads' success was immediate and New Orleans became a world port.

Improvement of the navigable depths in the passes can be correlated with new concepts in channel improvement, such as the European concept of jetties that was used by Eads and Corthell from 1875 to 1879 to improve South Pass for the passage of 30-ft draft ships. It can also be correlated with the devel-

[5] "History of the Jetties," by Corthell, 1881.

opment of dependable dredging equipment, such as the modern hopper dredge Langfitt, shown in Fig. 4, now used to maintain a 35-ft channel via Southwest Pass.

PREVIOUS IMPROVEMENT OF SOUTHWEST PASS AND SEA BAR CHANNEL

New Orleans port commerce increase rapidly following Eads' successful improvement of South Pass in 1875. By 1900 a deeper and larger channel through the passes was needed to accommodate the larger and deeper draft ships bound

FIG. 3.—ADVANCE—SOUTHWEST PASS SEA BAR

to and from New Orleans. In 1902 the Congress authorized the project for improvement of Southwest Pass to secure a 35 ft by 1,000 ft channel, as recommended by a 1899 board, at an estimated cost of $6,000,000, with annual maintenance of $150,000. This project included construction of two jetties, sills across the major outlets and dredging. When this work was completed in 1908

FIG. 4.—DREDGE LANGFITT

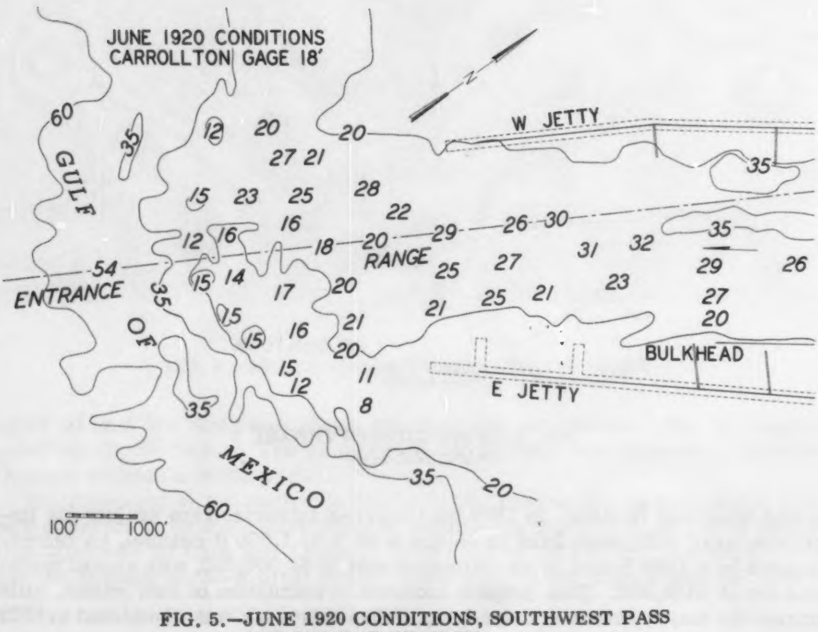

FIG. 5.—JUNE 1920 CONDITIONS, SOUTHWEST PASS
AND SEA BAR CHANNEL

the depth over the sea bar was increased from only 9 ft to 20 ft, far short of the expected 35 ft.

From 1911 to 1912 the east and west jetties of Southwest Pass were extended seaward 2,900 ft and 3,400 ft, respectively. The project dimension of 35 ft by 1,000 ft was not obtained by the jetty extension, but the natural depth was slightly increased.

After the 1911 to 1912 extension to the jetties, spur dikes were constructed in the lower 7 mi of the pass, contracting the width between dikes to about 3,000 ft. Again the desired 35-ft deep channel was not obtained.

Between 1917 and 1921 parallel inner bulkheads were constructed in the lower 5 mi, reducing the channel width from 3,000 ft to 2,400 ft.[6] In 1924 the east and west jetties were extended approximately 1,000 ft. The outer ends were turned channelward until they intersected the projected line of the inner bulkheads. Conditions at the mouth of Southwest Pass in June, 1920 are shown on Fig. 5. The controlling depth was then 16 ft over the sea bar.

From the start of the improvements to Southwest Pass in 1904 until 1921 all dredging on the sea bar was directed toward securing a channel directly across the sea bar along the prolonged axis of the jettied channel, as shown by Fig. 5, all with very limited success. In 1921 the axis of the sea bar channel was inclined 35° to the eastward of the axis of the jettied channel. The relocated sea bar channel is shown in Fig. 6. As in the case of South Pass, 25 yr before, this action was the determining factor in the subsequent success of providing the 35-ft deep channel across the sea bar.

The jettied channel was further contracted to lessen the maintenance dredging requirements. First, as noted previously, in 1921 the channel width was reduced from 3,000 ft to 2,400 ft. Then in 1923 and 1924 the width of the lower 8 mi was further reduced to 1,750 ft between dikes and the east and west jetties were extended. This contraction, combined with 7,000,000 cu yd of dredging, resulted in a 35-ft channel in the jettied channel and across the sea bar. February, 1924 conditions are shown in Fig. 6. Between 1937 and 1939 the lower 10 mi of Southwest Pass channel were contracted to 1,420 ft between dikes. Except during years of major floods a generally satisfactory 35-ft channel has been maintained in Southwest Pass since 1923. September, 1959 conditions in the lower jettied sector of Southwest Pass and the sea bar channel are shown in Fig. 7.

1956-57 DATA COLLECTION PROGRAM

A pilot model, constructed and tested in 1948 by the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, showed that the salt-water wedge and associated saline conditions in the passes could be duplicated and that a comprehensive model study was feasible. In fiscal year 1955 the Congress allocated $175,000 to initiate planning studies to provide the 40-ft deep channel via Southwest Pass, authorized by the R & H Act approved March 2, 1945. The Division Engineer, U. S. Army Engineer Division, Lower Mississippi Valley, Vicksburg, Miss., directed that the plan of improvement be based on prototype data, engineering and navigation studies and model investigations. A comprehensive data collection program was undertaken in Southwest Pass and Sea Bar by the New Orleans District to provide basic data for

[6] "Notes on the Mouth of the Mississippi River," by E. J. Dent, 1921.

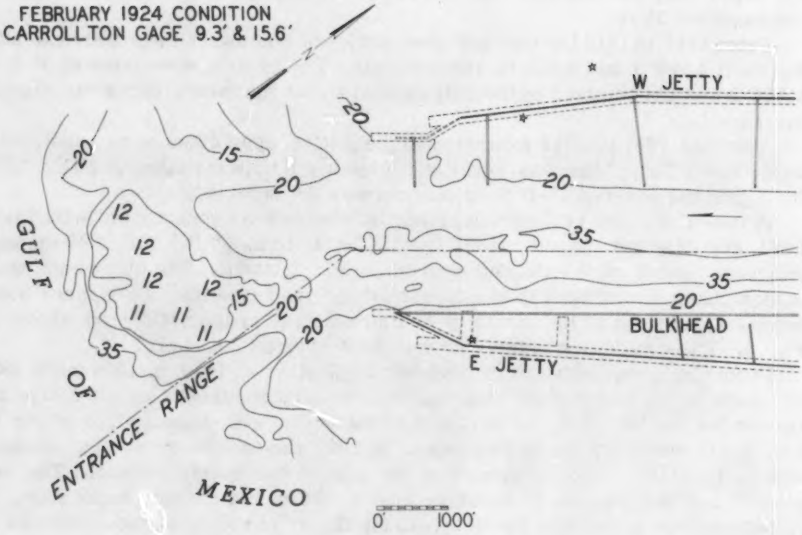

FIG. 6.—FEBRUARY 1924 CONDITIONS, SOUTHWEST PASS AND SEA BAR CHANNEL

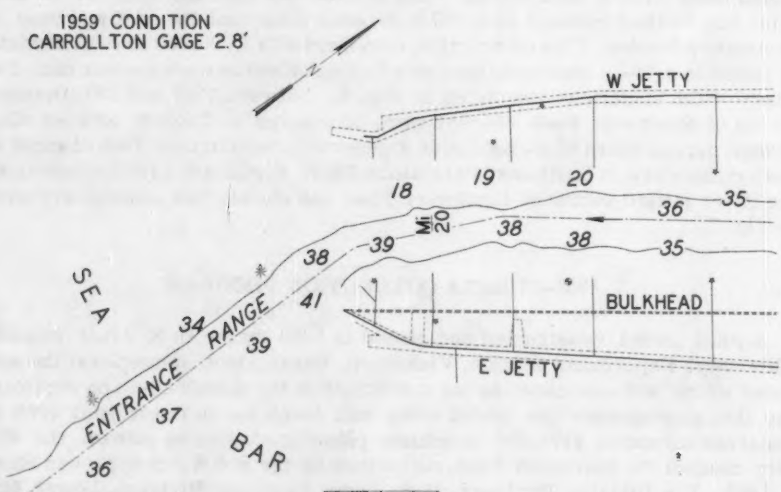

FIG. 7.—SEPTEMBER 1959 CONDITIONS, SOUTHWEST PASS AND SEA BAR CHANNEL

the prototype studies and data and information with which to plan, construct and verify a model of the lower 12 mi of Southwest Pass channel and Sea Bar channel. The data collection program was planned to obtain information on major factors influencing shoaling in tidal waters; namely the river's flow, including action of the salt-water wedge, channel alignment, and the Gulf tides. The field observations were made over a one year period to provide measurements during the complete hydrological cycle. Detailed data were obtained on the following: (1) flow in Southwest Pass (direction and magnitude); (2) salinity in pass channel and adjacent Gulf area; (3) sediment samples at various locations in the pass channel and adjacent Gulf area; (4) hydrographic surveys of channel and Gulf area; (5) Gulf flow currents; (6) tides; (7) winds; (8) waves; (9) past dredging requirements. (Data collected were compiled and published in two volumes by the New Orleans District in 1959.) The sectors of Southwest Pass and the Gulf covered in the data collection program are shown by Fig. 8. Fig. 9 depicts a marine stationary tower used in the program. Roberts radio current meter used in the program is shown in Fig. 10, and a recorder for a Roberts Meter in Fig. 11. Fig. 12 shows a small clamshell used to obtain bed material samples.

GULF TIDES

The Gulf tides are diurnal and have a range of 2 ft during spring tides. However, extreme tides caused by hurricane conditions have reached a stage of approximately 6 ft. At New Orleans, La. the average variation in low water river stage due to tides is approximately 10 in. to 12 in. During low water a slight tidal effect can be observed 35 mi above Baton Rouge, La.

FLOW CONDITIONS IN SOUTHWEST PASS

The fresh water discharge of the pass is controlled in part by Gulf tidal action. At low river stages the flow out of Southwest Pass may vary as much as 300%. Measurements showed a flow variation of 42,000 cfs to 128,000 cfs during one tidal cycle. At high river stages the tidal effect is minimized, and it may be ignored at flood stages. Normally, Southwest Pass carries 38% of the total river flow at the Head of Passes. A maximum flow of 354,000 cfs has been measured during flood conditions. An outflow of 24,900 cfs has been measured during low low-stage, slack water conditions. At this low stage the upstream flow of salt-water under the fresh-water layer was measured and found to be 25,900 cfs.[7] This upstream flow of Gulf water is hereinafter referred to as the "salt-water wedge."

SALT-WATER WEDGE PHENOMENON

Recorded measurements show that at extreme low stages the salt-water wedge may penetrate into the Mississippi River as much as 135 mi above the mouth of Southwest Pass, a fact which, at times, affected the New Orleans

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[7] "Investigations and Data Collection Program for Model Study of Southwest Pass, Mississippi River," New Orleans District, April, 1959.

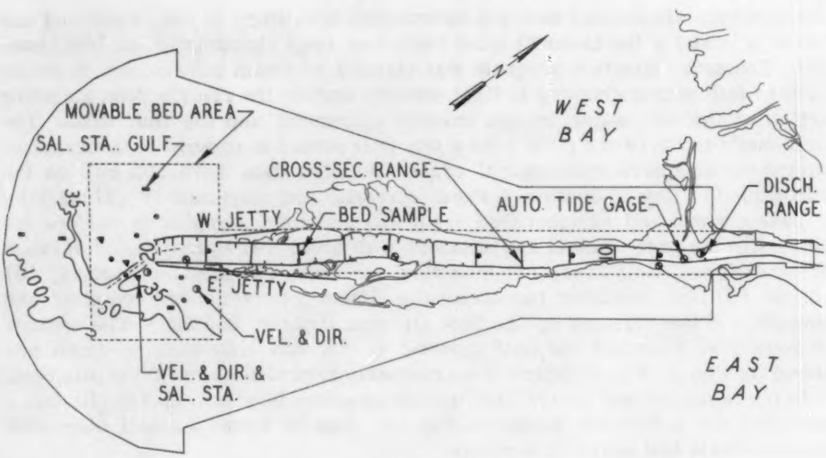

FIG. 8.—AREA COVERED IN DATA COLLECTION PROGRAM

FIG. 9.—MARINE TOWER USED IN DATA COLLECTION PROGRAM

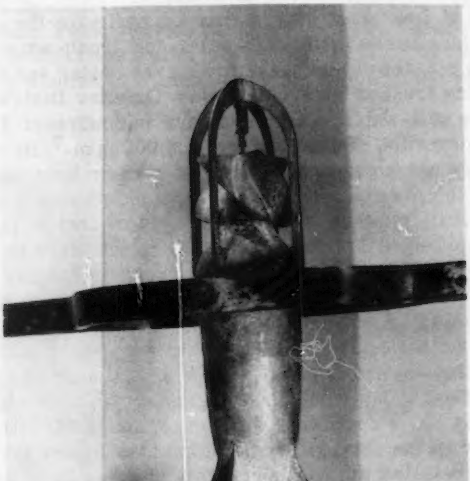

FIG. 10.—ROBERTS CURRENT METER USED IN DATA COLLECTION PROGRAM

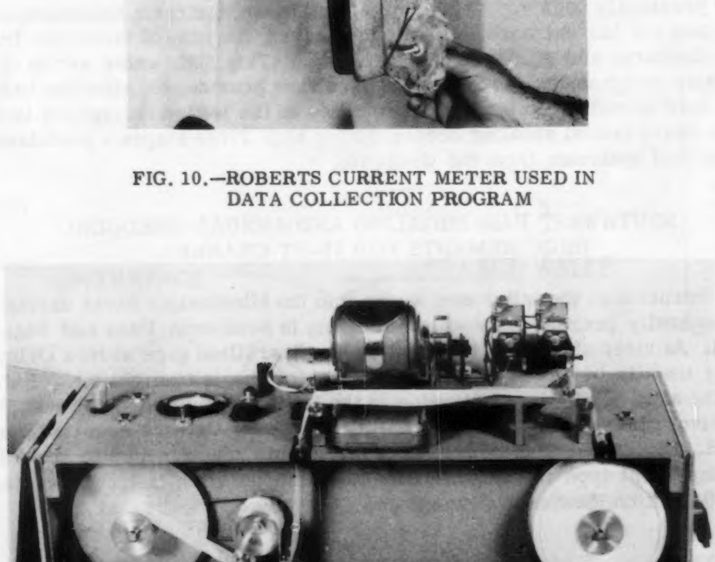

FIG. 11.—RECORDER FOR ROBERTS METER

water supply.[8] As previously noted, in Southwest Pass the upstream flow of salt-water wedge at low river stages may approximate the out-flow of fresh water. The interface between the salt-water and fresh-water is the turbulent layer between the masses of upstream flowing sea-water and the overlying and downstream flowing fresher water. The New Orleans District Office defines the location of the salt-water wedge interface in Southwest Pass as the plane at which the concentration of chloride ion is 5,000 ppm.[7] By contrast, the undiluted fresh-water has a concentration of 30 ppm or less, and the sea-water near the mouth of the pass about 20,000 ppm. In Southwest Pass the transition from low to high salinity takes place in only a few feet in the vertical. The locations of the wedge tip and the slopes of the interface in Southwest Pass channel were determined for fresh-water discharges ranging from 30,000 cfs to about 300,000 cfs in Southwest Pass.[7] Typical profiles of the salt-water wedge interface in Southwest Pass are shown in Fig. 13.

The vertical distribution of fresh-water and salt-water was determined and the current velocities were measured periodically at selected ranges in Southwest Pass during the 1956-57 field investigation.[7] Fig. 14 shows the vertical distribution at mile 19.92 Southwest Pass on January 27, 1957 at a low river stage. Fig. 15 shows the vertical distribution at a higher river flow on June 12, 1957. From a detailed study and analysis of these field data and his wide experience with tidal problems, Henry Simmons concluded, "***upstream velocities at given locations within the salt-water wedge are a function of the local slope of the interface, rather than a function of the entire wedge length as had previously been supposed, so that maximum upstream velocities in the jettied and sea bar entrance channel occur during the time of maximum fresh-water discharge and minimum wedge length." This high water action of the salt-water wedge under the fresh-water outflow provides an effective barrier to bed-load movement in the lower two miles of the jettied channel. It is here that the heavy annual shoaling occurs during high river stages - postulated to be rapid just upstream from the wedge tip.

SOUTHWEST PASS SHOALING AND ANNUAL DREDGING REQUIREMENTS FOR 35-FT CHANNEL

The intrusion of the salt-water wedge into the Mississippi River during low stage generally prevents low water shoaling in Southwest Pass and Sea Bar channel. At river stages 10 ft to 12 ft on the Carrollton gage at New Orleans, shoaling usually begins in the lower portion of the jettied channel. Fig. 16 shows the area of shoaling occurring in the jettied channel of Southwest Pass as the river rose from a low stage of 10 ft on the New Orleans gage on January 17, 1958. The annual hopper dredge requirements range from 4 to 10 million cu yd, dependent upon the magnitude of flow and duration of the river stages above 10-12 ft on the New Orleans gage.

PROTOTYPE STUDIES

It is axiomatic that engineers associated with the problems of improving Southwest Pass were guided by the work of Eads and Corthell in improving

[8] "Baton Rouge to the Gulf, Navigation Project," House Document No. 215, 76th Congress, 1st Session.

South Pass.[5] Although field investigations and the data collection program were not extended to South Pass, per se, the depth, with and alignment relations of both passes were analyzed and compared with attention to the sinuosity-depth relations. In these studies particular attention was addressed to a comparison

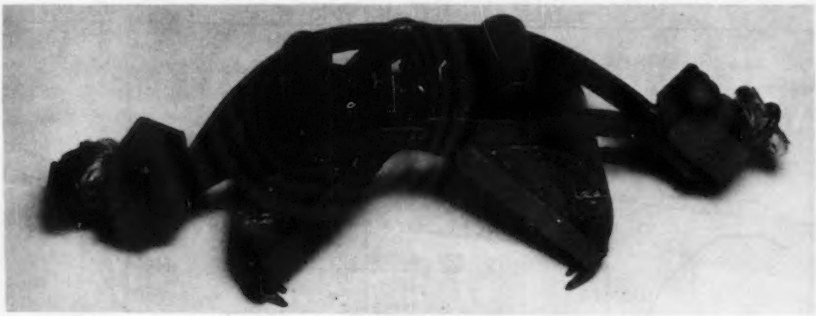

FIG. 12.—SMALL CLAMSHELL USED IN OBTAINING BED SAMPLES

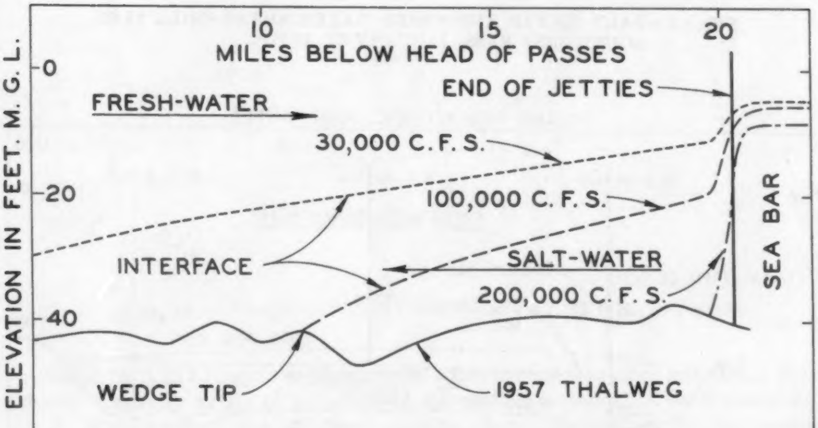

FIG. 13.—TYPICAL PROFILES, SALT-WATER WEDGE INTERFACE, SOUTHWEST PASS

of the geometry of the jettied sectors of the two passes and their sea bar channels.

Prototype studies included a re-examination of all official reports, articles and manuscripts by early and contemporary engineers associated with the improvement and maintenance of the passes, record survey maps, construction

records and the dredging history as concerns Southwest and South Passes. Prior investigations and concepts of the causes and methods devised for lessening the annual shoaling in Southwest Pass and Sea Bar channel were examined. Schultz' definitive sediment observations[9] and Dents' observations on

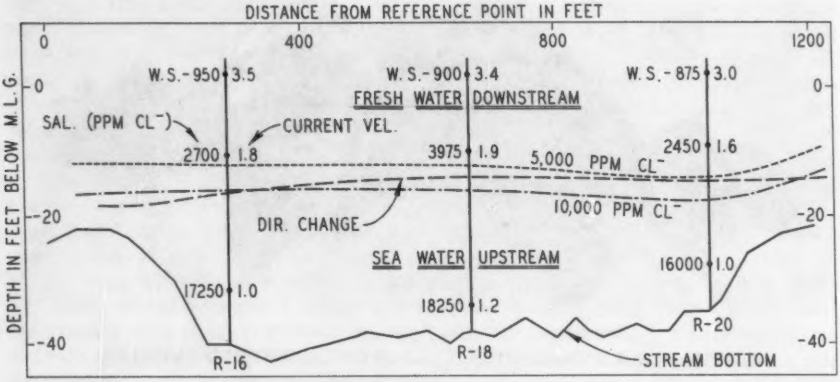

FIG. 14.—SALT-WATER AND FRESH-WATER AREAS—MILE 19.92 SOUTHWEST PASS, JANUARY 27, 1957

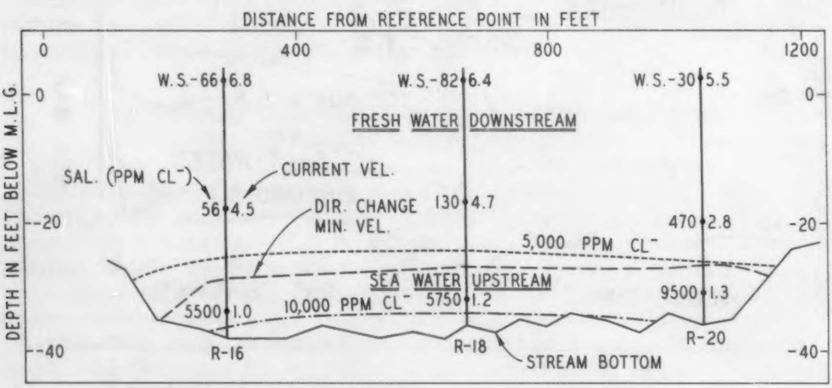

FIG. 15.—SALT-WATER AND FRESH-WATER AREAS—MILE 19.92 SOUTHWEST PASS, JUNE 12, 1957

9 "Sediment Observations, Passes of the Mississippi River," by Schultz, 1921.

the salt-water wedge[6] were correlated with the field investigations and data collection program.

GENERAL CONCLUSIONS FROM PROTOTYPE STUDIES AND FIELD INVESTIGATIONS

The following general conclusions were reached from the prototype studies and collating field observations:

1. Maintenance of a 40-ft channel through the existing Southwest Pass channel by dredging only would be difficult and expensive. It is estimated present hopper dredge requirements to maintain the 35-ft depth channel would be increased by 2 to 3 hopper dredge months. During major floods additional hopper dredge capacity would be required to maintain a dependable 40-ft channel.

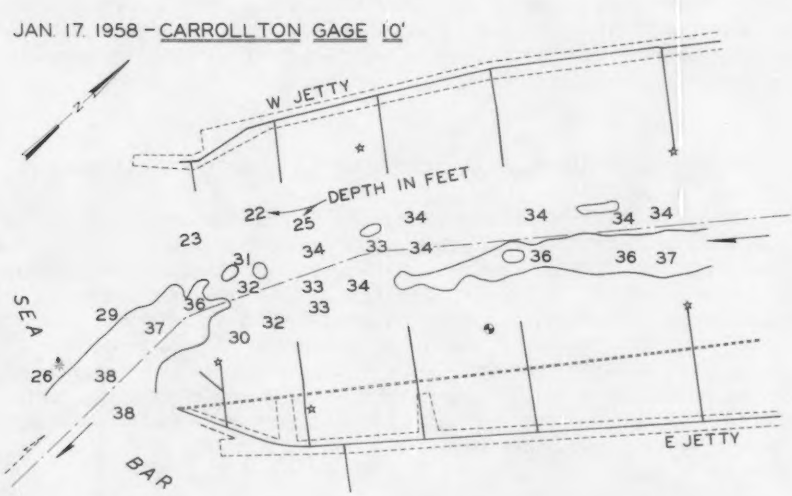

FIG. 16.—17 JANUARY 1958 CONDITIONS AT SOUTHWEST PASS

2. Premised on the excellent results of the Eads-Corthell improvement work at South Pass and an analysis of the existing channel alignment and channel areas, widths and depths of South and Southwest Passes, a more sinuous alignment will be required through the lower half of Southwest Pass, particularly below Mile 17 BHP, to secure a self-maintaining 40-ft ship channel.

3. During flood stages the Mississippi River carries a heavy bed load and a substantial sediment load. The existing high water flow regimen is favorable to deposition of the bed load and suspended sediment load materials in the lower sector and at the mouth of Southwest Pass.

4. During low water when the bed and suspended sediment load of the Mississippi River is greatly reduced, the upstream intrustion of the salt-water wedge seemingly acts as a shield against shoaling in the pass channel.

5. Under existing high water conditions the salt-water wedge acts as a barrier to bed-load movement beyond the wedge tip, and heavy shoaling may follow the wedge tip in its play upstream and downstream in the lower jettied channel and at the mouth of Southwest Pass, dependent upon fresh-water flows and tidal influences.

6. To be effective channel improvements must modify the high-water, fresh-water flow—salt-water wedge relation to alleviate the heavy high-water shoaling that now occurs in Southwest Pass. To provide a self-maintained channel through Southwest Pass, the channel improvement design must give more muscle to the high-water, fresh-water flow.

7. The law batture banks of the passes should be kept strong enough to prevent crevassing. They are subject to overflow to a depth of about one foot during floods. There is a slight annual subsidence of the batture banks, but river deposits during overbank flows are generally sufficient to maintain the batture banks to above mean sea level elevations. Where the batture is low and narrow the plan of improvement should provide for building it up with dredge spoil or strengthening by building up lands in the adjacent shallow bays by deposition from river flow through natural bayous and small, regulated outlets.

TWENTY-ONE POSSIBLE IMPROVEMENT SCHEMES EXAMINED

Engineers associated with the improvement and maintenance of the Mississippi River Passes presented 21 schemes for improving the lower three miles of Southwest Pass. These included a "dredging only" plan, and several plans proposing modifications to the existing structures, including extensions to jetties and extending the dikes to further contract the channel. Several schemes proposed some modification of the channel alignment. One scheme, Fig. 18, proposed to construct a new outlet and sea bar channel through the east jetty near mile 19 BHP. Fig. 17 shows one of the schemes for extending the jetties. Fig. 19 shows a scheme for increasing the sinuosity within the framework of the existing improvements below mile 17 BHP.

MODEL INVESTIGATIONS

The unique tidal model of lower Southwest Pass and Sea Bar channel was designed and operated at the U. S. Army Engineer Waterways Experiment Station by Messrs. H. B. Simmons and H. J. Rhodes, using the field data collected during 1957-58. The model tests provided excellent qualitative information on the shoaling problem. The fixed-bed model, scale 1: 500 horizontally and 1: 100 vertically, accurately reproduced the salt-water wedge phenomenon and duplicated the high-water shoaling pattern of the existing channel with colored plastic bed-load materials introduced at the upper end.

COMPLETE TESTS ON TWO PROPOSED PLANS

In model tests of the 21 improvement schemes, Plans 14 and 17A offered the best possibilities for lessening the heavy annual high water shoaling. Plan 14 is shown on Fig. 19. Plan 17A is shown on Fig. 18. These two plans were tested for conditions of river flow and Gulf tides representative of the full range

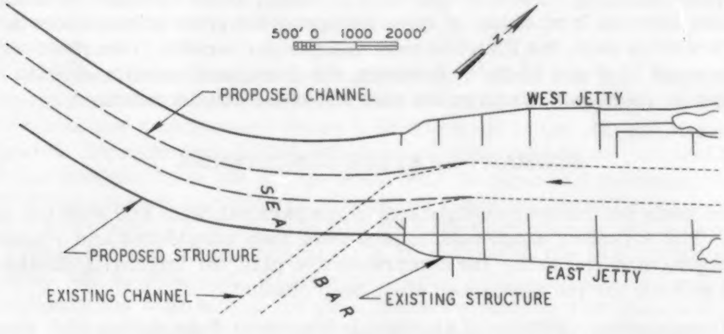

FIG. 17.—SCHEME FOR EXTENDING JETTIES

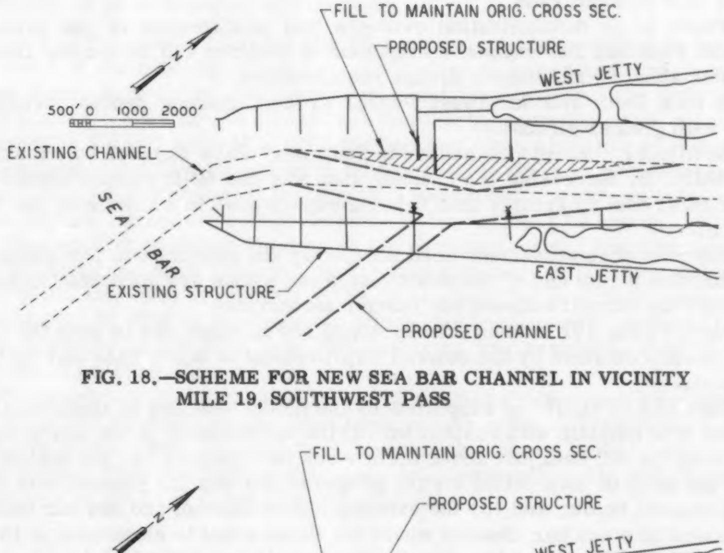

FIG. 18.—SCHEME FOR NEW SEA BAR CHANNEL IN VICINITY MILE 19, SOUTHWEST PASS

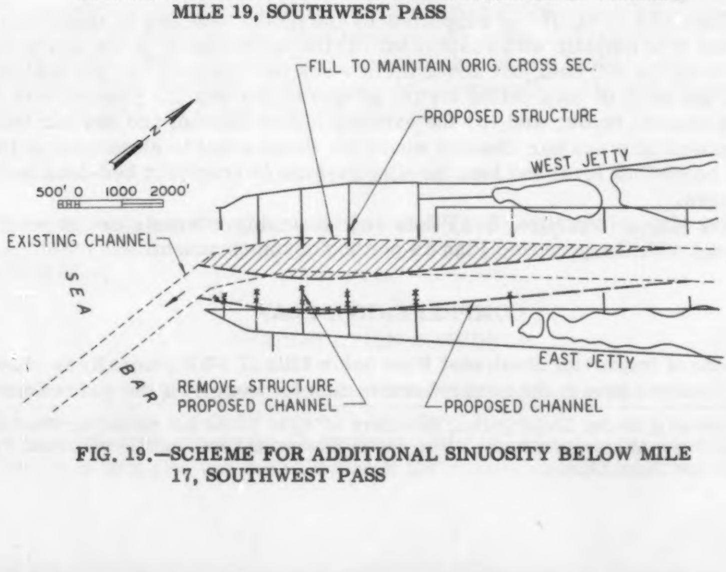

FIG. 19.—SCHEME FOR ADDITIONAL SINUOSITY BELOW MILE 17, SOUTHWEST PASS

that serious shoaling occurs in nature. The model tests indicated essentially equivalent benefits from either of these two plans for river stages above about 17 ft, Carrollton gage, but Plan 17A indicated greater benefits from river stages between about 10 ft and 16 ft.[10] However, the increased benefits of Plan 17A are offset by its higher construction cost and other considerations.

SUMMARY OF STUDY CONCLUSIONS

Model tests completed investigations of the physical facts and existing conditions. The following study conclusions were then considered and evaluated in developing and designing the comprehensive plan for improving Southwest Pass to provide for the authorized 40-ft deep channel:

1. The abnormal elevation of shoaling in Southwest Pass during high stages correlates closely with the transient locations of the salt-water wedge tip.
2. High water shoaling rarely occurs in South Pass channel, and annual dredging, if any, is nominal—seemingly the high fresh-water flow dominates the Gulf flow in South Pass.
3. There is no demonstration evidence that modification of the present Southwest Pass Sea Bar channel's alignment or location will lessen the annual high water shoaling and hopper dredge requirements.
4. In both South and Southwest Passes greater thalweg depths correlate closely with greater sinuosity.
5. Additional sinuosity is needed in Southwest Pass channel below about mile 9 BHP. By increasing the channel sinuosity the 40-ft channel should be maintained at cost no greater than is being experienced in maintaining the 35-ft channel.
6. The plan of improvement need not modify the existing sea bar channel. The velocities at the end of the Southwest Pass jetties are sufficient to keep the sea bar far enough seaward for reach maintenance.
7. Plan 14 (Fig. 19) is theoretically sound and is supported by both the excellent results obtained by the channel improvement at South Pass and by the model tests.
8. Plan 17A (Fig. 18) is supported by the model test and is theoretically sound, but it is quixotic with respect to: (a) the maintenance of the sharp curvature along the left bank just above the new sea bar channel; (b) the maintenance of the point of bifurcation for the proposed new sea bar channel with the existing channel below; and (c) the existing jettied channel and sea bar below the proposed new sea bar channel would not doubt shoal to elevations of 10 ft to 15 ft below Gulf level and lose its effectiveness to transport bed-load during high stages.
9. The Mississippi River Bar Pilots and steamship interests are in general agreement with the proposed Plan 14 method of improvement.

COMPREHENSIVE PLAN

The plan of improving Southwest Pass below Mile 17 BHP generally as shown in Fig. 19 was adopted in the comprehensive plan for improving the entire South-

10 "Hydraulic Model Investigation, Summary of Best Plans for Reducting Shoaling, Southwest Pass, Mississippi River," Waterways Experiment Station Miscellaneous Paper No. 2-349, July, 1959.

west Pass channel. The structures, dredging and other construction required to provide the 40-ft channel via Southwest Pass are:

Dredging.—

1. Between mile 1.1 AHP in the Mississippi River and mile 17.5 BHP, the 40-ft channel (with 2 ft overdepth) will be obtained by dredging where existing dimensions are deficient and where a modification in the channel alignment is required. An estimated 13,000,000 cu yd of material will be removed by cutterhead dredge. Here the dredge spoil will be deposited overbank on either side of the channel to raise and strengthen areas of low and narrow batture.

2. Between mile 17.5 and mile 20.2 BHP the 40-ft channel will be excavated by cutterhead and hopper dredges. It is estimated that 4,600,000 cu yd of dredging will be required. Where practicable, cutterhead dredge spoil may be deposited on the west side of the pass channel within the dike system to retain, insofar as possible, the existing cross-sectional area of the channel. Otherwise, the spoil will be placed on the east side of the channel.

3. The 40-ft by 600-ft sea bar channel running from Mile 20.2 BHP to the 40-ft contour in the Gulf of Mexico will be obtained by hopper dredging. Some 2,300,000 cu yd of dredging will be required.

Dikes.—The existing lateral pile dike system will be modified by lengthening some of the existing dikes, by construction of new dikes and removal of some dikes. A new dike system may be required along the right bank between mile 2 and mile 3 BHP, to contact and realign the channel in this reach. Between mile 9.0 and mile 20.2 BHP the existing dikes along the west bank will be lengthened and new dikes added. The portion of the dike work between mile 9.0 and mile 12.3 is designed to strengthen and protect the narrow batture banks from crevassing and wave wash erosion. Between mile 12.3 and mile 20.2 BHP the dike work is designed to promote the development and stabilization of the channel along the more sinuous alignment, particularly in the reach below mile 17.5 BHP. Removal of portions of the dikes along the east bank between mile 18 and mile 20 BHP is necessary to effect this realignment. A total length of approximately 13,000 ft of new dike extension are required. Approximately 3,000 ft of existing dikes will be removed.

Bulkheads.—A longitudinal bulkhead of the Wakefield type will be constructed to protect the badly eroded west bank between mile 15.3 and mile 16.1 BHP.

Relocations.—The proposed realignment of the channel in the lower two miles of Southwest Pass will require the relocation of navigation aids, and minor modifications of the U. S. Coast Guard facilities.

ESTIMATE OF COST

The total estimated Federal cost of the project is $8,300,000, not including associated Federal cost of $140,000. Non-Federal first cost for relocations is $18,000.

CONSTRUCTION SCHEDULE

It is estimated that construction will require approximately 3-1/2 years. The dredging work at the entrance to Southwest Pass will be conducted in two phases. In the first phase the 35-ft channel will be shifted to the new easterly alignment by dredging and modification of the channel contraction dikes. Deep-

ening of the channel to 40 ft will be accomplished in the second phase. The longitudinal bulkhead between miles 15 and 16 BHP will be constructed in the early part of the construction period. Construction of other training works and structures in the pass will be initiated in the latter part of the construction period.

BENEFITS

The trend of modern shipping is toward larger, deeper draft vessels. Channel depths in excess of 40 ft exist from a short distance above Head of Passes to New Orleans. The 40-ft channel will permit ships to load to deeper drafts, thus utilizing all cargo space, and will eliminate the hazards of deep draft vessels navigating in Southwest Pass due to fluctuation in depths in the pass. The latest approved benefit-to-cost ratio for the project, Mississippi River, Baton Rouge to the Gulf of Mexico, La., is 16.7 to 1 (May 1958). This ratio as currently estimated is 18.5 to 1, due to increased waterway tonnage. Benefits due to the incremental increase in channel depth from -35 to -40 ft between New Orleans and the Gulf of Mexico were evaluated separately.

ACKNOWLEDGMENTS

The Southwest Pass improvements are in accordance with the plan recommended by Col. Cookson, District Engineer, New Orleans, and Gen. Carter, Division Engineer, Vicksburg, and approved by Gen. Itschner, Chief of Engineers, Corps of Engineers.

The author is indebted to a number of people in the New Orleans District, the Waterways Experiment Station and the Lower Mississippi Valley Division for advice, suggestions and in preparing the paper. He particularly wishes to acknowledge the advice and suggestions of Messrs. Rhodes, Simmons, Gentilich, Kennedy, Toffaleti, McCleave, Miss Jones and Mrs. McQuiller.

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DISCUSSION

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LABORATORY INVESTIGATION OF RUBBLE-MOUND BREAKWATERS[a]

Discussion by Thorndike Saville, Jr.
Closure by Robert Y. Hudson

THORNDIKE SAVILLE, JR.[1]—The author has ably presented a quantity of useful data leading to continued progress toward a sounder basis for stability and economic design of rubble-mound structures. In the course of the extensive testing carried out, the author has also obtained a great deal of valuable data pertaining to wave run-up on rubble slopes. These data are presented in Figs. 8 through 10 in which relative run-up (R/H) has been plotted as a function of the wave steepness (H/λ) determined for the water depth at the structure toe. As the wave steepness for any particular wave train critically depends on the depth of water for which it is determined (varying by a factor of as much as 3 or 4, for ordinary waves), the plotted points have also been segregated according to relative depth (d/λ). However, the wave steepness range covered for each relative depth tested is generally small, and, as the author notes, the scatter induced by measurement difficulties and general complexity of action is large, so that the true effect of relative depth is obscured. The author has, accordingly, drawn a near-envelope curve, and indicates that he feels it should be used for determining design values.

Unfortunately, since values of both height (H) and length (λ) depend on the relative depth in which they are measured, use of such a curve under the assumption that it is independent of relative depth produces different values of run-up for any particular wave train, depending on where the wave characteristics are measured. An illustrative example is given in Table 1, where values of wave run-up (R) as determined from Fig. 9 for a 1 on 2.5 slope are tabulated for a single wave train moving from deep water into shallow. The tabulations are made for arbitrarily selected water depths for which the wave characteristics are determined. The particular wave train chosen has a height and period of 8 ft and 6 sec in a 10-ft water depth. In making the computations, Fig. 9 was used as though it were completely independent of relative depth.

It will be noted that, in this case, predicted run-up of 5.9 ft is obtained if the wave characteristics, as measured in 10 ft of water, are used. However, if the designer had chosen to determine his wave characteristics at a depth of 25 ft instead, he would have obtained a predicted run-up of 6.8 ft. And if he were working with deep water characteristics, he would have determined a 7.8 ft predicted run-up. As the table shows, a different value of run-up will

[a] September, 1959, by R. Y. Hudson.

[1] Asst. Chf., Research Div., Beach Erosion Bd., U. S. Army Corps of Engrs., Washington, D. C.

be determined for each relative depth value for which the wave characteristics may be measured. Actually, however, we know that a wave incident on a given structure must have only one value of run-up for a particular shore condition.

It would seem, therefore, that run-up curves such as these should be referred to a specific relative depth rather than being considered as independent of relative depth. Fortunately, there are readily available methods of determining heights and lengths for any depth, or relative depth, if the height and length are known at some particular depth. Wiegel, for example, has tabulated[2] ratios of height and length to their deep water values as a function of relative depth. Using these tables, the author's data for each relative depth may be transformed into equivalent data for a single selected reference relative depth. The particular relative depth selected as reference is immaterial, and one is probably as good as another. However, a deep-water-reference relative depth has previously been used[3,4] and has some advantages since deep water values are frequently the actual known values, and the tabulated ratios relate to deep water values.

TABLE 1.—RUN-UP COMPUTATIONS, 1 ON 2.5 SLOPE

Depth (d) (ft) (1)	Height (H) (ft) (2)	Period (T) (Sec) (3)	Length (λ) (ft) (4)	$\frac{d}{\lambda}$ (5)	$\frac{H}{\lambda}$ (6)	$\frac{R^a}{H}$ (7)	R^a (ft) (8)	$\frac{R^b}{H}$ (9)	R^b (ft) (10)
10	8.00	6.0	101.5	.0986	.0788	.74	5.9	.80	6.4
15	7.56		120.6	.124	.0627	.84	6.4	.84	6.4
18.45	7.40		130.9	.141	.0565	.89	6.6	.86	6.4
25	7.26		145.8	.172	.0497	.93	6.8	.88	6.4
36.9	7.28		163.9	.225	.0444	.96	7.0	.89	6.5
184.5	7.93		184.5	1.00	.0430	.98	7.8	.81	6.4

[a] From Hudson, Figure 9.
[b] From present discussion, Figure 3c.

Accordingly, the author's run-up data for the 1 on 2.5 slope have been referred to a deep water basis $(d/\lambda = 1.0)$, and are replotted in Fig. 1. The scales of the graph have been changed to logarithmic (rather than to arithmetic, as the author used) to stretch out the points at the lower steepness values. Such stretching seems to indicate a possible tendency for relative run-up to decrease with decreasing steepness below a certain critical steepness value—although this conclusion is largely dependent on the location of but a single point.

All plotted points in this figure now refer to a single relative depth, and a mean curve could be drawn through them. However, the scatter is still large, and a curve somewhat above the mean has been drawn. (This curve is also plotted in Fig. 1 and 3(c), on arithmetic scales.)

2 "Gravity Waves, Tables of Functions," by R. L. Wiegel, Council on Wave Research, Engrg. Foundation, 1954.

3 "Wave Run-up on Shore Structures," by Thorndike Saville, Jr., Proceedings, ASCE, 1956, (also Transactions, ASCE, 1958).

4 "Wave Run-up on Roughened and Permeable Slopes," by R. P. Savage, Proceedings, ASCE, Vol. 84, No. WW 3, 1958, (also Transactions, 1959).

Similar curves (also referred to a deep water relative depth, ($d/\lambda = 1.0$) have been drawn for the other slopes for which the author gives data. These are shown as a family of curves in Fig. 2. Also shown in this figure, as a matter of comparison, are similar curves for smooth slopes as derived from previously published reports.[3,4,5,6] The roughness and permeability of the rubble had an obvious reducing effect! These curves are shown in terms of H_o'/T^2 (where T is the wave period) rather than H_o'/L_o purely as a matter of personnel convenience, because the deep water wave length, L_o, equals $5.12\ T^2$, the two ratios bear a constant relationship, with one being essentially five times the other.

As these curves are all referred to a deep water relative depth, the designer, in using them, must compute the deep water wave steepness and height from whatever design wave information he has, and use these values in computing the run-up. Similar curves referred to any other relative depth could,

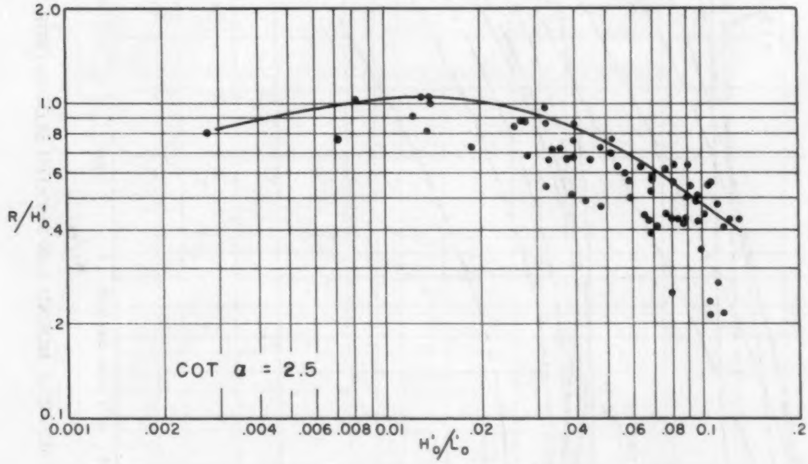

FIG. 1.—WAVE RUN-UP ON RUBBLE SLOPE, 1 ON 2.5 DEEP WATER REFERENCE RELATIVE DEPTH

of course, be drawn, and would be equally usable. However, in using them, one must know the relative depth of reference, and be careful to use wave characteristics for that particular reference relative depth. This is true even though this reference depth may not physically exist in the actual field location (as "deep water" does not for many lakes, for example). In this way, one always gets the same run-up computed for a given wave and structure. For the case tabulated previously, and using the curve shown in Fig. 2, this run-up value would be 6.4 ft.

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5 "Wave Run-up on Composite Slopes," by Thorndike Saville Jr., Proceedings, Sixth Internatl. Conf. on Coastal Engrg., Council on Wave Research, Engrg. Foundation, 1958.

6 "Shore Protection Planning and Design," Beach Erosion Bd., Tech. Report Number 4, U. S. Army Engr. Beach Erosion Bd., Washington, D. C.

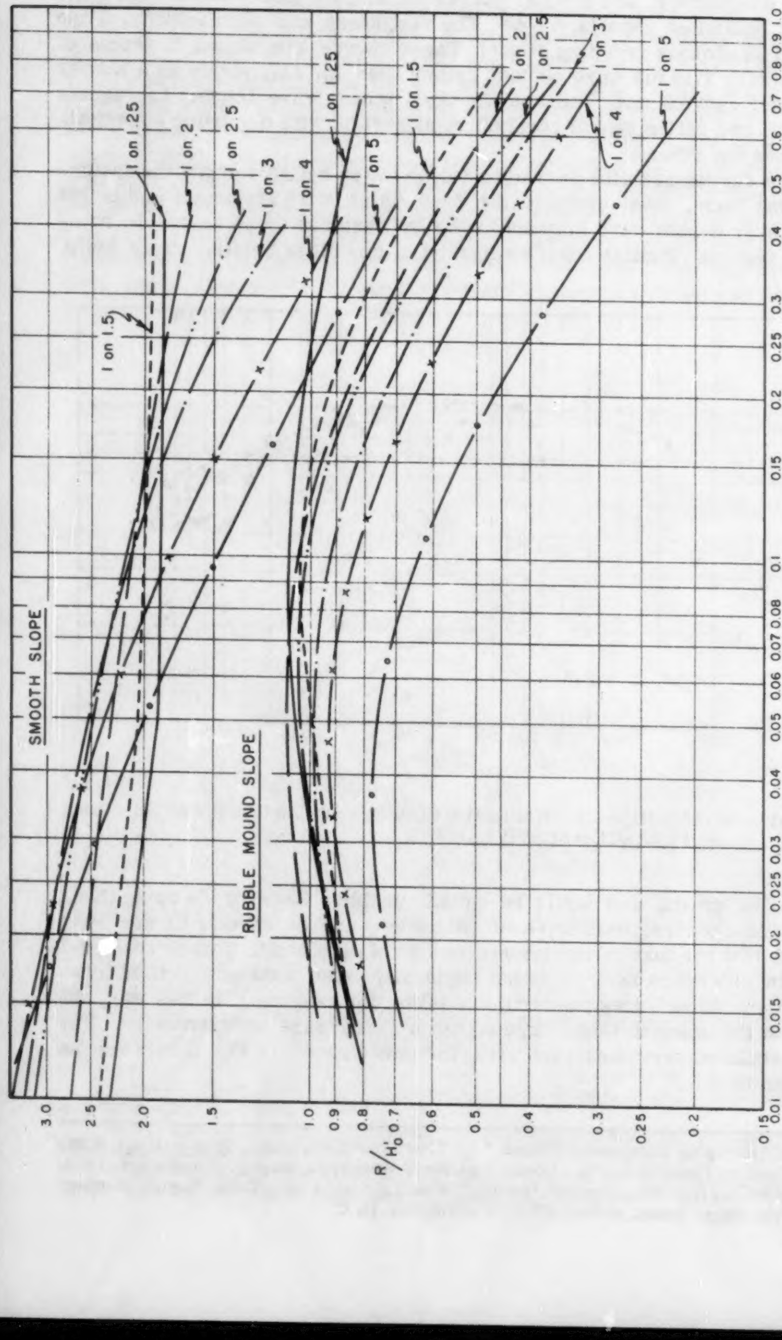

FIG. 2.—WAVE RUN-UP ON RUBBLE MOUND AND SMOOTH SLOPES (FOR VALUES OF $d/H'_o > 3$)

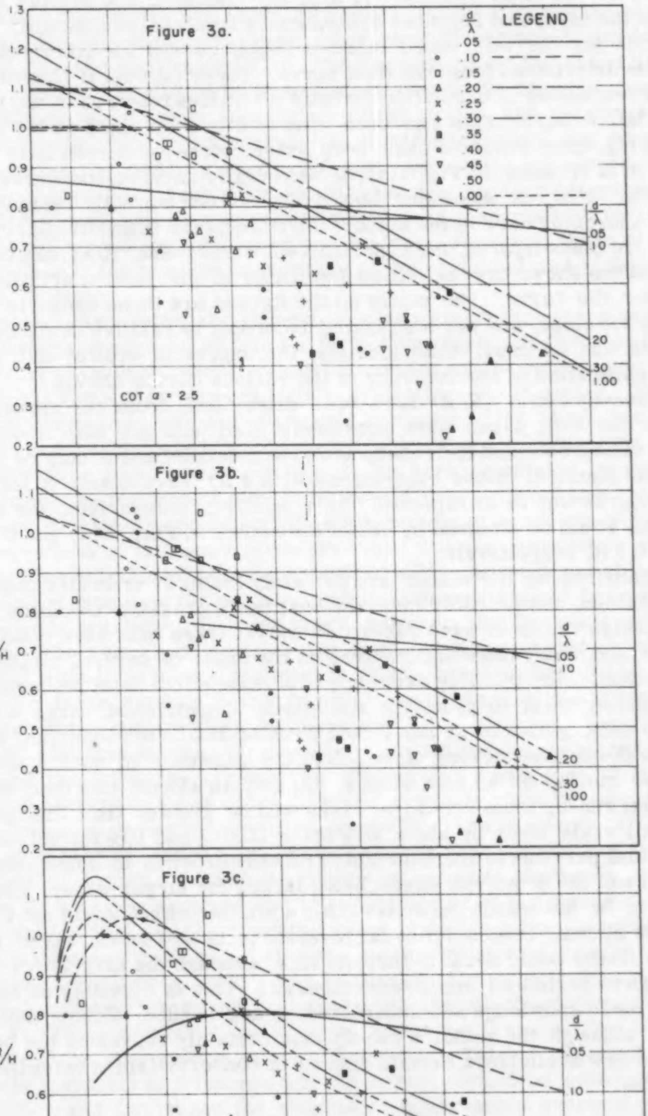

FIG. 3.—WAVE RUN-UP ON RUBBLE SLOPE, 1 ON 2.5. RELATIONSHIP TO RELATIVE DEPTH FOR SEVERAL POSSIBLE RUN-UP CURVES

If, because of the large quantities of data to be handled, one prefers not to have to make the many and repeated computations involved in obtaining wave characteristics for the referenced relative depth, curves for other relative depths may be determined from the first curve. These curves, if plotted as a family of curves, enable interpolation directly for values referred to any other reference relative depth. This has been done in Fig. 3 for the 1 on 2.5 slope data. In so doing, three different basic deep water reference curves have been drawn. The first of these, in Fig. 3(a), is essentially an envelope curve; the second, in Fig. 3(b), is somewhat less conservative but still retains the straight line character used by the author; the third is the (logarithmic) curve from Fig. 1. In these figures, the abscissa (wave steepness, H/λ) values for each of the curves shown are then those applicable to the particular relative depth (d/λ) for the curve. The points on the figures are those shown in Fig. 9 for the 1 on 2.5 slope, and are segregated according to relative depth by the same symbols that he used. Unfortunately, the degree of scatter still prevents a good estimation of the accuracy of the various curves shown.

Values of run-up factor (R/H) have been determined from the curves in Fig. 3(c) for the 8-ft, 6-sec wave previously used, and are also shown in Table 1. The values obtained for run-up are also tabulated and it may be seen that essentially identical values (approximately 6.4 ft) were obtained for all cases, as of course was to be expected. As a matter of comparison, the run-up values which would be obtained by use of the curves in Figs. 3(a) and 3(b), were 7.1 and 6.5 ft, respectively.

The data gathered by the author are certainly the most extensive and inclusive yet obtained, and probably form the best basis presently available for determining design values of wave run-up. However, these data were obtained in small scale laboratory tests using waves on the order of several inches in height. Accordingly, the possible existence of a scale effect must be borne in mind when applying them to prototype conditions. Unpublished, large scale (2-5 ft waves) data, gathered at the Beach Erosion Board in connection with design of Lake Okeechobee levees, have shown the existence of such a scale effect for wave run-up on smooth slopes. On smooth slopes, this data indicates that actual run-up from prototype waves will be greater than that predicted by small scale tests by about 20% for a 1-on-3 and 10% for a 1-on-6 slope. The actual percentage increase also probably depends to some extent on the exact size of the prototype waves, being larger for larger waves. There would appear to be no reason to believe that a similar effect would not also exist for rubble slopes. Indeed, a few large-scale tests now under way at the Beach Erosion Board would seem to indicate this, although the data is not yet complete enough to permit an exact determination. This is certainly an area in which much more knowledge and understanding are needed. Consequently, it is felt that, although the author's run-up data certainly represent the best basis for design now available, a certain amount of conservatism in using them may be advisable.

ROBERT Y. HUDSON.[1]—The extensive discussions of this paper are much appreciated. The varied backgrounds and the experience of the discussors make their contributions especially significant. Although the discussion by Messrs, Carvalho and Vera-Cruz will probably be appreciated more by those

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[1] Chf., Wave Action Sect., U. S. Army Engr. Waterways Experiment Sta., Vicksburg, Miss.

engaged in laboratory tests of rubble breakwaters, design engineers in America are also indebted to these gentlemen for the information presented concerning the full-scale breakwater from which Iribarren obtained the data used to compute the coefficient (K'). The discussions by Messrs. Slichter, Booth, and Lillevang are valued because they represent the ideas of practicing engineers with many years experience in the design and construction of rubble breakwaters. Mr. Jones' discussion provides excellent but provisional data concerning the stability of riprap cover layers for fill slopes. The stability of cover layers for rubble breakwaters and the stability of riprap cover layers for fill slopes are closely related phenomena, especially from the standpoint of laboratory testing techniques and analyses. However, there is considerable difference in the relative importance of the variables, and in the practical aspects of design.

In each discussion, the thought was expressed that the science of rubble breakwater design is still in the formative stage, and that considerable work remains to be done before stability formulas can be relied upon for exact solutions. The writer concurs in this evaluation of the status. Yet, he cannot agree completely with Mr. Slichter's comment to the effect that "although the guide lines contributed by the author's work are valuable aids, the design of breakwaters must remain largely in an area dominated by sound judgment based on observations of past experience." It is believed that the observations of past experience should include observations of small-scale tests as well as observations of full-scale structures, and that the formulation of sound judgment should be based both on the relationships between variables, determined by use of small-scale models, and on lessons learned from observation of full-scale breakwaters. It is agreed, however, that the designer cannot afford to use the results of small-scale tests blindly, and that, after all possible model tests have been completed on rubble breakwaters, there will still remain a large area in design in which use of the intuition and practical experience of the designer will be necessary.

Mr. Slichter doubts that there is any practical design significance in abandoning the Iribarren formula, in view of the small size of the armor units tested relative to those of a full-scale structure. The reasons for abandoning the Iribarren formula were in no way related to the effects of model scale on the accuracy of test results. It is believed that the scale of the tests is sufficient to insure an adequate degree of dynamic similarity, model to prototype, if it can be assumed that the shape factor of the armor units, the placing of armor units, and the characteristics of incident wave trains, are sufficiently similar. It is hoped that the testing programs, now in progress at the Corps of Engineers, U. S. Army Waterways Experiment Station and the Beach Erosion Board laboratory, can determine the effects of these variables.

For those who favor the original Iribarren formula (Eq. 1 of the author's paper), the comparison of results shown in Table 1, obtained by substitution in the formulas of Iribarren and the author, should be informative. In Table 1 $(W_r)_I$ and $(W_r)_H$ are the weights of quarrystone armor units required for stability for the different slopes, assuming equal wave heights, obtained by substituting in the Iribarren and Hudson formulas, respectively. Values of 0.015 for K' and 3.2 for K_Δ were used. It is noted from this comparison that (a) the formulas agree for a slope of 1-on-2, (b) the weights of quarrystone armor units required for stability, as determined from the Iribarren formula, are approximately one-half those obtained from the author's formula when the

slope is 1-on-3 or flatter, (c) the effects of small changes in μ are very large for steep slopes, and (d) the weights of armor rock given by the Iribarren formula for slopes of 1-on-1-1/2 and 1-on-1-1/3 are exceptionally high, compared with the author's formula, especially for low values of μ.

Mr. Slichter questions the assumption, made in developing the general functional equation (Eq. 13), that friction between armor units can be neglected. This assumption is, of course, not strictly correct, but it is believed sufficiently accurate for use in developing the functional equation. This belief is based on several years of experience in testing small-scale rubble breakwaters to the point of failure. It was concluded from observing these tests that friction between armor units can be a primary resisting force for pellmellplaced units only when the entire cover layer is on the verge of sliding downslope. It was noticed that, for pellmellplaced armor units, there is considerable failure of individual units, by forces that lift and roll the units out of their nested positions, before failure occurs by sliding of the cover-layer mass. This is not true, however, for the condition where armor units are carefully selected for shape and are positioned individually to obtain maximum wedging action. For this condition, especially for the steeper slopes, the cover layer usually fails by sliding along the boundary between the cover-layer bottom and the first underlayer. It is hoped that the effects of shape factor and special placing techniques can be determined in future tests.

TABLE 1.—COMPARISON OF THE IRIBARREN AND HUDSON FORMULAS

cot α	$(W_r)_I/(W_r)_H$		
	μ = 1.00	μ = 1.05	μ = 1.10
1-1/3	8.0	5.4	3.9
1-1/2	3.4	2.6	2.1
2	1.1	1.0	0.8
3	0.6	0.5	0.5
4	0.5	0.5	0.5
5	0.5	0.5	0.5

Messrs. Carvalho and Vera-Cruz indicate the following beliefs; (a) the stability of rubble breakwaters situated in relatively shallow water on sandy bottoms is affected by the increased specific gravity of the water resulting from the great amounts of sand stirred up by the waves; (b) the inertia force is important and should be investigated; (c) in the analytical basis of the stability equation, an effort should be made to express the vectorial character of the wave forces; (d) the most adequate criterion for the nodamage condition should be one in which only the active portion of the breakwater, and one or two layers of the protective cover, are taken into account; (e) safety factors should be determined for each type of breakwater section; (f) Iribarren's formula is susceptible of experimental verification, if it is assumed that $\mu = 1$, for all slopes flatter than 1-on-1, and provided that K' is not considered constant; and (g) K_Δ should not be called a shape coefficient. The writer agrees with the ideas expressed in items (d) and (e). The work explained in the original paper represents the first phase of a comprehensive testing program, and it is agreed that the criterion used for the no-damage tests, and the damage or safety-factor data, are applicable directly only to the type of sections

used in the tests. Tests in progress (1960) compare damage to the cover layer with the volume of armor units in the cover layer itself.

The idea expressed in item (a) is new to the writer, and it should no doubt be investigated. It is predicted, however, that tests will show that the stirring-up of sand in the water by wave action does not appreciably affect the stability of rubble breakwaters. The question as to whether the forces of inertia (Eq. 3) are important, concerns a very complicated phenomena, and the writer is not qualified to argue the point. However, the success attained in correlating the test results, using the derived functional equation, in which the effects of inertia forces were included in the experimental coefficient K_{Δ}, indicates that the assumption made is sufficiently accurate. Also, the work of McNown and Keulegan,[2] concerning the relation of drag and inertia forces on flat plates and cylindrical bodies in periodic motion, appear to indicate that the time available for the formation and shedding of vortexes is not sufficient, in the flow around breakwater armor units caused by short-period wave action, to result in large inertia forces. It is doubted that an attempt to express the vectorial character of the wave forces in the functional equation would be successful (item c above). The direction of the wave forces vary with d/λ, H/λ, H/d, β, σ, $\cot \alpha$, h/H, and t/T. Thus, it was decided to omit the direction variable from the general stability equation, and let the model define the importance and the effects of its function. The success achieved to date in correlating the test results using Eq. 13 appear to substantiate the correctness of this decision.

The writer agrees that Iribarren's formula is susceptible to experimental verification if it is assumed that $\mu = 1$. However, this is tantamount to assuming that friction is important at the time the formula is derived, then assuming it is unimportant for the purpose of experimental verification. It is believed better to let the model tests define the effects of friction. Also, Iribarren's formula includes the term involving the breakwater slope ($\cos \alpha - \sin \alpha$), which originated from an assumption with respect to the directions of both wave forces and friction forces. The inadequacy of this assumption is believed to be the primary reason that K' varies so considerably with breakwater slope.

The information presented by Messrs, Slichter and Booth, concerning the design of breakwaters in Hawaii using tribars and tetrapods, should be very useful to other designers. It is good to know that the test results, obtained during the model study conducted at the Waterways Experiment Station to determine the optimum design for repairing the Nawiliwili Harbor breakwater, have been partly verified by the prototype breakwater's ability to resist the action of large storm waves.

This breakwater was designed, based on the model test results, to be stable for waves as large as 24 ft in height. The storm referred to by Mr. Booth was hurricane "Dot," which passed directly over Kauai, the island on which the Nawiliwili breakwater is situated. Although an accurate estimate of the heights of waves that attacked the breakwater is not presently available, photographs taken before the height of the storm show considerable overtopping of the breakwater. By comparison with similar photographs taken during the model study, it is estimated that the waves were greater than 20 ft in height.

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[2] "Vortex Formation and Resistance in Periodic Motion," by J. S. McNown and G. H. Keulegan, Proceedings, ASCE, Vol. 85, No. EM 1, January, 1959.

The writer agrees with Mr. Booth's statement that values of the experimentally determined coefficient K_Δ, presented in the author's paper, should not be considered as final. However, the information presented by Mr. Booth concerning the Nawiliwili Harbor breakwater and the information presented by Mr. Deignan[3] concerning the Crescent City breakwater, indicate that the model test results can be used to predict the action of prototype structures with very good accuracy. The large-scale tests being conducted by the Beach Erosion Board for the Waterways Experiment Station, mentioned by Mr. Booth, are not complete at this time, but preliminary results indicate that the author's values of K_Δ for quarrystone are conservative.

The writer agrees with Mr. Lillevang that the placing of armor units in such a way as to effect wedging action and interlocking between adjacent units will increase the stability of rubble breakwaters, compared with the stability obtained by pell-mell placement. However, maximum benefit of especially placed armor units cannot be obtained unless all units down to an elevation equal to about -H ft, referred to swl, are so placed. The reason for this is that hydrostatic pressure of water in the cover layer is considerable, and is maximum at an elevation immediately above the trough of the wave on the structure. If the special placing of armor units is carried to the extreme, voids in the cover layer may be decreased sufficiently to cause failure due primarily to the hydrostatic pressure of water trapped in the cover layer. The test data presented in Table 2 answers Mr. Lillevang's request for information concerning the effects of shape factor on the stability of cover layers.

TABLE 2.—EFFECTS OF ARMOR UNIT SHAPE FACTOR ON STABILITY OF BREAKWATER STEM IN RELATIVELY DEEP WATER[a]

Armor Unit (1)	n (2)	Placement (3)	K_Δ (4)	k_Δ (5)	P, percentage (6)
Quarrystone	2	Pell-mell	2.6	1.0	40
Tetrapod	2	Pell-mell	8.3	1.0	50
Tribar	2	Pell-mell	12.0	1.0	54
Tribar	1	Regular	25.0	1.1	47
Tetrahedron	2	Pell-mell	3.0	1.0	40
Quadripod	2	Pell-mell	8.3	1.0	50

[a] $\cot \alpha \leq 1.5$

These data were obtained using water of considerable depth, relative to wave height, and for the no-overtopping condition. The results of all tests to date are provisional to the extent that the effects of scale, variations in wave height (heights $> H_{1/3}$) in storm-wave trains, placing techniques, overtopping waves, waves breaking directly on the structure, and the angle of wave attack have not been determined. However, when waves do not break directly on the structure, and when there is no overtopping, it is believed that use of the values of K_Δ in the Table 2 will result in conservative designs. It is believed, also, that the relative economy of rubble breakwaters, similar except for the

3 "Breakwater at Crescent City, California," by John E. Deignan, Proceedings, ASCE, Vol. 85, No. WW 3, September, 1959.

shape of armor unit used in the cover layer, can be determined with considerable accuracy, using the writer's formula and the above values of K_Δ, k_Δ, and P.

The information presented by Mr. Jones concerning the measured value of the friction coefficient (μ) is valuable design data. The average value of μ = 0.75, determined for dumped riprap, would increase the required stone weights nearly four times for cot α = 2, as explained by Mr. Jones, compared with the use of μ = 1.0 and K' = 0.015 in Iribarren's formula. It should be realized, however, that the value of K' = 0.015 should not be used in Iribarren's formula for riprap cover layers. A new value of K' for riprap would need to be determined, using, in this determination, the new value of μ = 0.75 for the friction factor in Iribarren's formula.

Mr. Saville has correctly pointed out that both wave height (H) and wave length (λ), and thus wave steepness (H/λ), depend on depth of water in which H and λ are determined, and that different values of run-up (R) are determined from the author's run-up curves, depending on just where the wave characteristics are measured. However, this fact does not detract from the usefulness or accuracy of the author's run-up curves, as stated by Mr. Saville. In the paper, H was defined as the wave height measured at the location of the proposed breakwater. Thus, there should be no confusion as to which value of H/λ to use, because there is only one value of H/λ corresponding to a selected breakwater position and a given deep-water design wave H_0/λ_0. The dimensions of a deep-water design wave are usually selected on the basis of existing hindcasting techniques, and the corresponding value of H/λ, for depth (d) of the selected breakwater location, is determined by constructing wave-refraction diagrams for the area between deep water and the proposed structure location. It is believed that the method of plotting the run-up data selected by the author is not only correct from an analytical standpoint, but also is more convenient and less confusing than the curves proposed by Mr. Saville. Also, the tests from which the run-up data were obtained were conducted using water of considerable depth relative to wave height (H/d varied from a minimum of 0.28 to a maximum of 0.35).

Thus, it is doubtful whether the data can be used for the condition of breaking waves, as did Mr. Saville in his example (an 8-ft wave of 6-sec period will break in a water depth of about 10.5 ft). It is hoped that future tests can be conducted for the condition of gradually increasing depth seaward of the breakwater location, using both breaking and nonbreaking waves, and a larger range of values of d/λ. We are indebted to Mr. Saville for pointing out the possible effects of scale on wave run-up. The writer was not aware that scale effects for run-up on rubble breakwaters were appreciable. Until more data are available concerning scale effects on wave run-up, the crowns of breakwaters probably should be designed to withstand some over-topping, without failure, whenever the run-up curves shown in the paper are used.

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

WAVES IN NAVIGATION CANALS DUE TO LOCK FILLING[a]

Discussion by F. F. Escoffier

F. F. ESCOFFIER,[5] M. ASCE.—The author has presented the basic theory of canal waves and the results of the model studies carried out by him in substantiation of this theory in an excellent manner. The writer will try to present certain considerations in regard to this theory that will be helpful in the understanding of the waves and in devising numerical solutions to some problems.

Eq. 2 was apparently obtained by substituting the mean depth of the water into a formula that had been originally derived for waves in a rectangular channel. The writer is not prepared to say whether this, in itself, has a significant effect on the accuracy of the formula. However, he would like to point out that it can be misleading to regard the original formula as an exact equation even in a rectangular channel. He will also try to show that the Lagrange formula properly interpreted, is the more accurate formula.

The original formula[4] is

$$C = \sqrt{g\left(D_1 - \frac{3}{2}\,y + \frac{1}{2}\,\frac{y^2}{D_1}\right)} \quad \ldots\ldots\ldots\ldots \quad (49)$$

in which C is the velocity of propagation of the surge, D_1 is the initial depth, g is the acceleration of gravity, and y is the difference between the depths upstream and downstream from the surge. This formula applies to negative surges, however, by changing the sign of the quantity y it can be made to apply to positive surges.

When applied to a positive surge, it indicates a loss of energy that is appropriate in the case of a travelling jump but not otherwise. When applied to a negative surge it indicates a gain in energy, which is impossible. The last statement is simply another aspect of the well known fact that negative jumps entail an increase in energy and are accordingly impossible. The resolution of the difficulty lies in the fact that, except for travelling jumps, surges are not uniformly progressive waves but undergo deformation as they advance in the channel. It is, therefore, meaningless to assign a single celerity to a surge unless it is a travelling jump. To avoid possible, confusion the writer will point out that uniformly progressive waves are possible in channels where the effect of friction is significant. However, this has no bearing on the present discussion.

[5] Hydr. Engr., U. S. Army, Corps of Engrs., Mobile, Ala.

The dynamic equation for a non-uniform channel can be written in the form

$$g \frac{\partial h}{\partial x} + \frac{\partial v}{\partial t} + \alpha v \frac{\partial v}{\partial x} + (1-\alpha) \frac{Wv}{A} \frac{\partial h}{\partial t} = -g \frac{Q^2}{K^2} \quad \ldots\ldots \quad (50)$$

in which A is the cross-sectional area, h is the elevation of the water surface, K is the conveyance, Q is the discharge, t is time, x is distance measured in a downstream direction, and α is the Boussinesq velocity-distribution coefficient. This is essentially the equation given by Einstein and Fuchs.[6] The equation of continuity is

$$\frac{\partial Q}{\partial x} + W \frac{\partial h}{\partial t} = 0 \quad \ldots\ldots \quad (51)$$

If the dependent variables h and v in the partial derivatives of Eq. 50 and 51 are replaced by the dependent variables H and Q where

$$H = h + \alpha \frac{v^2}{2g} \quad \ldots\ldots \quad (52)$$

and

$$Q = A\,v \quad \ldots\ldots \quad (53)$$

These equations become

$$\frac{\partial H}{\partial x} + \frac{\frac{N^2}{W}\frac{\partial Q}{\partial t} - \alpha v \frac{\partial H}{\partial t}}{c^2 - \alpha v^2} = -\frac{Q^2}{K^2} \quad \ldots\ldots \quad (54)$$

and

$$\frac{\partial Q}{\partial x} + \frac{g A \frac{\partial H}{\partial t} - \alpha v \frac{\partial Q}{\partial t}}{c^2 - \alpha v^2} = 0 \quad \ldots\ldots \quad (55)$$

where

$$c = \sqrt{\frac{g A}{W}} \quad \ldots\ldots \quad (56)$$

and

$$N = \sqrt{1 + (\alpha - 1)\,\alpha \frac{v^2}{c^2}} \quad \ldots\ldots \quad (57)$$

If Eq. 55 is multiplied by $\pm \frac{N}{c\,W}$ and then added to Eq. 54 the resulting equation, with some rearrangement of terms, is

$$\frac{\partial H}{\partial x} \pm \frac{N}{c\,W} \frac{\partial Q}{\partial x} + \frac{\frac{\partial H}{\partial t} \pm \frac{N}{c\,W} \frac{\partial Q}{\partial t}}{\pm N c + \alpha v} = -\frac{Q^2}{K^2} \quad \ldots\ldots \quad (58)$$

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6 "Computation of Tides and Tidal Currents, United States Practice," by H. A. Einstein and R. A. Fuchs, Proceedings, ASCE, Vol. 81, June, 1955.

The left side of Eq. 58 is a directional derivative in the x, t plane. If the upper sign is taken it is a derivative along the path defined by

$$dx = (N\,c + \alpha v)\,dt \quad \ldots\ldots\ldots\ldots\ldots \quad (59)$$

and the equation can be written in the form

$$dH + Z\,dQ = -\,\frac{Q^2}{K^2}\,dx \quad \ldots\ldots\ldots\ldots\ldots \quad (60)$$

where

$$Z = \frac{N}{c\,W} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (61)$$

Eqs. 59 and 60 are the equations for the forward characteristic and the corresponding equations for the backward characteristic are

$$dx = (-\,N\,c + \alpha v)\,dt \quad \ldots\ldots\ldots\ldots\ldots \quad (62)$$

and

$$dH - Z\,dQ = -\,\frac{Q^2}{K^2}\,dx \quad \ldots\ldots\ldots\ldots\ldots \quad (63)$$

From Eqs. 59 and 62 it can be seen that the celerity of a wave element is

$$\omega = \pm\,N\,c + \alpha v \quad \ldots\ldots\ldots\ldots\ldots \quad (64a)$$

which, for the usual assumption that $\alpha = N = 1$, becomes

$$\omega = \pm\,c + v \quad \ldots\ldots\ldots\ldots\ldots \quad (64b)$$

As c is the Lagrangian velocity, defined by Eq. 56, and as no simplifying assumptions have been introduced into the foregoing derivations, it is clear that the Lagrange formula is an exact rather than an approximate guide to the celerity of wave elements.

An interesting variation of Eqs. 60 and 63 is obtained by introducing as dependent variables

$$U = \frac{H}{\epsilon} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (65)$$

and

$$V = \epsilon\,Q \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (66)$$

where

$$\epsilon = \sqrt{Z} = \sqrt{\frac{N}{c\,W}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (67)$$

The variables Z, U, V, and have been called the "surge impedance," "head transformation," "flow transformation," and "root surge impedance," respectively, by Paynter and Ezekial.[7] They show these concepts to apply to such diverse physical phenomena as transients in electric transmission lines, seismic waves in the ground, vibration in turbine shafts, tidal waves in estuaries,

[7] "Water Hammer in Nonuniform Pipes as an Example of Wave Propagation in Gradually Varying Media," by H. M. Paynter and F. C. Ezekiel, Transactions, ASME, Vol. 80, No. 7, October, 1958.

acoustic propagation in horns, and water hammer. According to Paynter and Ezekiel these concepts were introduced by Evangelisti[8] and Schelkunoff.[9] The modified equations are

$$d(U + V) = -(U - V)\frac{d\epsilon}{\epsilon} - \frac{Q^2}{\epsilon K^2}\,dx \qquad (68)$$

which applies to the forward characteristic and

$$d(U - V) = -(U + V)\frac{d\epsilon}{\epsilon} - \frac{Q^2}{\epsilon K^2}\,dx \qquad (69)$$

which applies to the backward characteristic.

The new dependent variables have the advantage of being less variable in nonuniform channels than the original ones. Furthermore, the new equations are better adapted to the application of Picards method of numerical solution.

In the foregoing derivations, the writer has sought to retain as high a degree of generality as practicable and accordingly, has introduced no simplifying assumptions. Therefore, Eqs. 59 and 62 together with either Eqs. 60 and 63 or Eqs. 68 and 69, are mathematically equivalent to Eqs. 50 and 51. Hence, any solution which satisfies one set of equations will also satisfy the others. At this point, it is assumed, in order to obtain results comparable to those of the author, that $\alpha = 1$ and $K = \infty$. The first assumption implies $N = 1$ and Eqs. 59, 60, 61, 62, 63, 67, 68, and 69 become

$$dx = (c+v)\,dt \quad \text{(forward characteristic)} \qquad (70)$$

$$dH + Z\,dQ = 0 \quad \text{(forward characteristic)} \qquad (71)$$

$$Z = (cW)^{-1} \qquad (72)$$

$$dx = (-c+v)\,dt \quad \text{(backward characteristic)} \qquad (73)$$

$$dH - Z\,dQ = 0 \quad \text{(backward characteristic)} \qquad (74)$$

$$\epsilon = (c\,W)^{-\frac{1}{2}} \qquad (75)$$

$$d(U + V) = -(U - V)\frac{d\epsilon}{\epsilon} \quad \text{(forward characteristic)} \qquad (76)$$

$$d(U - V) = -(U + V)\frac{d\epsilon}{\epsilon} \quad \text{(backward characteristic)} \qquad (77)$$

In a uniform horizontal channel Eqs. 71 and 74 can be reduced to the forms

$$dr + dv = 0 \quad \text{(forward characteristic)} \qquad (78)$$

$$dr - dv = 0 \quad \text{(backward characteristic)} \qquad (79)$$

in which r is a stage variable defined by the equation

$$r = \int_0^h \frac{g\,dh}{c} \qquad (80)$$

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8 "Sul calcolo del colpo D'ariete nelle condotte forzate a caratteristoche variabili," by G. Evangelisti, L'Energia Elettrica, Vol. 16, December, 1939.

9 "Remarks Concerning Wave Propagation in Stratified Media," by S. A. Schelkunoff, Symposium on the theory of electromagnetic waves, Interscience Publishers, Inc., New York, 1951.

From Eqs. 78 and 79, it can be inferred that $r + v$ has a constant value along a forward characteristic and $r - v$, a constant value along a backward characteristic. This leads to a fairly simple and accurate solution for an upper pool or a canal having an unlimited length and a constant width as assumed by the author. However, the rather restrictive assumption of a uniform width is not essential as any uniform cross section can be assumed.

In the lock-filling operation considered by the author, the quantity given at the upstream boundary, that is, at the lock is Q. As $r - v$ has a constant value along a backward characteristic the following can be written

$$r - v = r_o - v_o \qquad (81)$$

where r_o and v_o are the initial or undisturbed values of r and v. Furthermore

$$Q = - A v \qquad (82)$$

By eliminating v between Eqs. 81 and 82 we obtain

$$Q = A (r_o - v_o - r) \qquad (83)$$

Every quantity on the right side of Eq. 83 is either a constant or a function of h and it is, accordingly, possible to compute Q for any desired value of h. If Q is then plotted against h, it becomes possible to read off the value of h for any desired value of Q.

If a pool or canal of finite length is to be considered the Schnyder-Bergeron graphical method can be used. This is illustrated in Fig. 26, which represents a pool or canal terminating at its downstream end in a large pool or lake having a constant water level. The curve ab shows the lowering of the water level in the canal at its downstream end due to velocity head. It is obtained by plotting r against the velocity

$$v = -\sqrt{2 g (h_o - h)} \qquad (84)$$

in which h_o is the initial value of h. The curves cd, ef, and gh are obtained by plotting r against the velocity

$$v = - \frac{Q_n}{A} \qquad (85)$$

in which Q_n is the value of Q after a lapse of time sufficient for a wave element to travel from the lock to the downstream end of the canal and return n times.

In a transition or in any other nonuniform channel, the foregoing method cannot be used and it becomes necessary to devise a numerical solution involving either Eqs. 71 and 74 or Eqs. 76 and 77. Numerical methods that apply to Eqs. 71 and 74 have been devised by Schonfeld[10] and described in detail by him. The present discussion will be confined to Eqs. 76 and 77 because they are better adapted to the application of Picard's method and also because they will throw considerable light on the accuracy of Green's law.

It will be assumed that the initial flow is zero and that the datum to which the water surface elevation and the total head are referred is the initial undisturbed water level. The initial values Q_o and H_o are, therefore, zero. In

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[10] "Propagation of Tides and Similar Waves," by J. C. Schönfeld, Staatsdrukkerij-En Uitgeverijbedrijf 'S-Gravenhage, 1951.

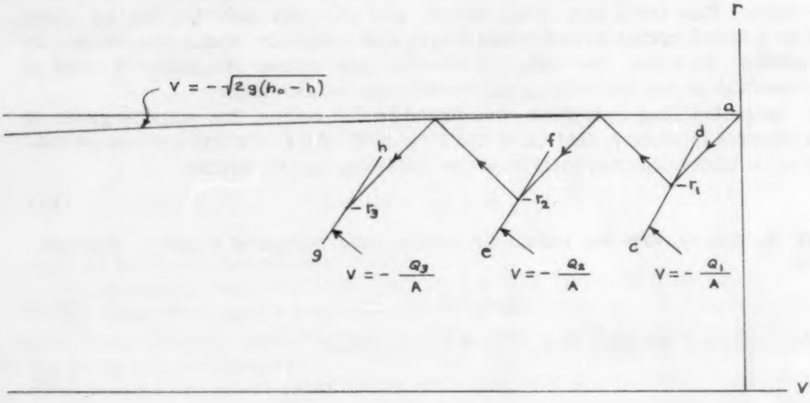

FIG. 26.—APPLICATION OF SCHNYDER-BERGERON METHOD TO UNIFORM CHANNEL

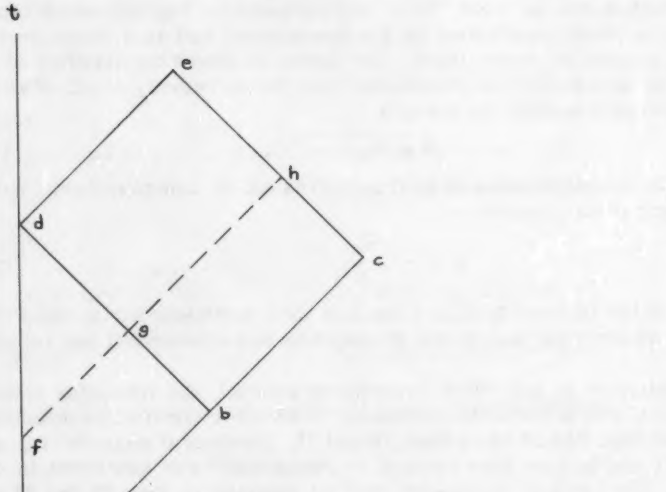

FIG. 27.—DISTANCE-TIME DIAGRAM FOR NONUNIFORM CHANNEL

Fig. 27, the three forward characteristics abc, fgh, and de, and the two backward characteristics bgd and che have been drawn, the flow of water into the lock, Q, is given as a function t along the left-hand boundary. The area below the line abc represents the initial undisturbed water. From a consideration of Eqs. 76 and 77 the following can be written

$$(U_e + V_e) - (U_d + V_d) = -\int_d^e (U - V) \frac{d\epsilon}{\epsilon} \quad \ldots\ldots \quad (86)$$

$$(U_d - V_d) - (U_b - V_b) = -\int_b^d (U + V) \frac{d\epsilon}{\epsilon} \quad \ldots\ldots \quad (87)$$

$$(U_e - V_e) - (U_c - V_c) = -\int_c^e (U + V) \frac{d\epsilon}{\epsilon} \quad \ldots\ldots \quad (88)$$

Picard's method consists essentially in replacing these equations with the iterative formulas

$$(U_{e,n} + V_{e,n}) - (U_{d,n} + V_{d,n}) = -\int_d^e (U_{n-1} - V_{n-1}) \left(\frac{d\epsilon}{\epsilon}\right)_{n-1} \quad \ldots \quad (89)$$

$$(U_{d,n} - V_{d,n}) - (U_{b,n} - V_{b,n}) = -\int_b^d (U_{n-1} + V_{n-1}) \left(\frac{d\epsilon}{\epsilon}\right)_{n-1} \quad \ldots \quad (90)$$

$$(U_{e,n} - V_{e,n}) - (U_{c,n} - V_{c,n}) = -\int_c^e (U_{n-1} + V_{n-1}) \left(\frac{d\epsilon}{\epsilon}\right)_{n-1} \quad \ldots \quad (91)$$

which indicate how the nth approximation is to be obtained from the (n-1)st approximation. In the present application, a first approximation is conveniently obtained by assuming that the integrals on the right hand side of Eqs. 86, 87, and 88 are negligible in value. By keeping in mind that

$$U_b = V_b = U_c = V_c = 0 \quad \ldots\ldots\ldots\ldots\ldots \quad (92)$$

these equations are seen to become

$$U_e + V_e = U_d + V_d \quad \ldots\ldots\ldots\ldots\ldots \quad (93)$$

$$U_d - V_d = 0 \quad \ldots\ldots\ldots\ldots\ldots \quad (94)$$

$$U_e - V_e = 0 \quad \ldots\ldots\ldots\ldots\ldots \quad (95)$$

from which it is inferred that

$$U_e = V_e = U_d = V_d \quad \ldots\ldots\ldots\ldots\ldots \quad (96)$$

In view of Eqs. 65 and 66 this can be rewritten

$$\frac{H_e}{\epsilon_e} = \epsilon_e Q_e = \frac{H_d}{\epsilon_d} = \epsilon_d Q_d \quad \ldots\ldots\ldots\ldots\ldots \quad (97)$$

from which is derived

$$H_e = \epsilon_d \epsilon_e Q_d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (98)$$

$$Q_e = \frac{\epsilon_d}{\epsilon_e} Q_d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (99)$$

$$H_e = \frac{\epsilon_e}{\epsilon_d} H_d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (100)$$

Eqs. 98 and 99 are the desired equations. Paynter and Ezekial[4] refer to Eqs. 99 and 100 as the "generalized Green's Law."

If the foregoing first approximation is substituted into the right hand side of Eqs. 89, 90, and 91 there are obtained the equations

$$(U'_e + V'_e) - (U'_d + V'_d) = 0 \quad \ldots\ldots\ldots\ldots\ldots (101)$$

$$(U'_d - V'_d) - 0 = -2 \int_b^d \epsilon_f \, Q_f \left(\frac{d\epsilon}{\epsilon}\right)_g \quad \ldots\ldots\ldots\ldots (102)$$

$$(U'_e - V'_e) - 0 = -2 \int_c^e \epsilon_f \, Q_f \left(\frac{d\epsilon}{\epsilon}\right)_h \quad \ldots\ldots\ldots\ldots (103)$$

where the primes have been used to indicate the second approximations and the subscripts f, g, and h show the location of the variables in Fig. 27. With the aid of Eqs. 65 and 66 the following equations are obtained from Eqs. 101, 102, and 103.

$$\frac{H'_e}{\epsilon'_e} = \epsilon'_d \, Q_d - \int_b^d \epsilon_f \, Q_f \left(\frac{d\epsilon}{\epsilon}\right)_g - \int_c^e \epsilon_f \, Q_f \left(\frac{d\epsilon}{\epsilon}\right)_h \quad \ldots (104)$$

$$\epsilon'_e \, Q'_e = \epsilon'_d \, Q_d - \int_b^d \epsilon_f \, Q_f \left(\frac{d\epsilon}{\epsilon}\right)_g + \int_c^e \epsilon_f \, Q_f \left(\frac{d\epsilon}{\epsilon}\right)_h \quad \ldots\ldots (105)$$

Eqs. 104 and 105 provide a clue to the accuracy of Green's Law. It can be seen at once that if a wave is short the paths of integration bd and ce will also be short and the corrections to be applied to Green's Law will be small. By evaluating the integrals the errors can be estimated. However, the evaluation of the integrals as a rule requires some ingenuity. Only the particular case will be considered here in which the height of the waves is small enough in comparison with the depth of the water to justify the assumption that the celerities c + v and c - v remain equal to the initial value c_o and that the root surge impedance ϵ remains equal to its initial value ϵ_o. As c_o and ϵ_o are functions of x alone, it becomes possible to introduce the new variables

$$\tau = \int_o^x \frac{dx}{c_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots (106)$$

and

$$\nu = \frac{co}{\epsilon o} \frac{\partial \epsilon o}{\partial x} = \frac{1}{\epsilon o} \frac{\partial \epsilon_o}{\partial \tau} \quad \ldots\ldots\ldots\ldots (107)$$

Accordingly Eqs. 70 and 73 can be rewritten

$$d\tau = dt \quad \ldots\ldots\ldots\ldots\ldots\ldots (108a)$$

and

$$d\tau = dt \quad \ldots\ldots\ldots\ldots\ldots\ldots (108b)$$

respectively, from which it can be inferred that τ - t will be constant along a forward characteristic and τ + t will be constant along a backward characteristic. It is now possible to rewrite Eqs. 104 and 105 as

$$H'(\tau,t) = \epsilon_o(\tau)\ \epsilon_o(0)\left\{Q(o,t-\tau)+\frac{1}{2}\int_o^{t-\tau} Q(o,\theta)\left[\nu\left(\frac{t-\tau-\theta}{2}\right)+\nu\left(\frac{t+\ -\theta}{2}\right)\right]d\theta\right\} \quad \ldots(109)$$

$$Q'(\tau,t) = \frac{\epsilon_o(0)}{\epsilon_o(\tau)}\left\{Q(o,t-\tau) + \frac{1}{2}\int_o^{t-\tau} Q(o,\theta)\left[\nu\left(\frac{t-\tau-\theta}{2}\right)- \nu\left(\frac{t+\tau-\theta}{2}\right)\right]d\theta\right\} \quad \ldots(110)$$

The writer has made no numerical calculations to compare these methods with those of the author and is, therefore, not in a position to discuss their relative accuracy. He believes that the Schnyder-Bergeron graphical method can be used to great advantage in many problems relating to waves in canals and other channels. The substitution of the total head for the static head in Green's Law results in a more accurate formula although the effect will probably be negligible in the waves studies by the author. By evaluating the integrals in Eqs. 104 and 105 or in Eqs. 109 and 110 it is possible to arrive at an estimate of the error in Green's Law.

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

MEAN DIRECTION OF WAVES AND OF WAVE ENERGY[a]

Discussion by J. W. Dunham, Per Bruun, and J. M. Jordaan, Jr.

J. W. DUNHAM,[2] F. ASCE.—Although important advances toward the understanding of shore processes have been made in recent years, the effect of wave action on littoral sands, in areas where man interferes with nature, still defies accurate prediction. The author's approach to the particular problem with which he is dealing is ingenious and may shed additional light on one more facet of the complex field of coastal technology. Many factors must be taken into consideration, however, when one attempts to predict the movement of littoral material in a changed environment. The energy-vector concept, as presented in the paper, certainly appears to apply at Santa Barbara. The writer questions, however, whether this type of analysis will result in reliable solutions to similar problems elsewhere under widely varying conditions.

It may be wise first to study certain analogous examples of natural shorelines. Wherever littoral material is moved from a beach that generally parallels the breaker crests to a downdrift segment of coast that veers sharply away from the average alignment of these crests, the mode of movement of the littoral material seems to change from a broad sluggish band to a narrow swiftly moving stream fairly close to shore. If the coastline is steep and rocky in this segment, this littoral stream may go well below the water surface only to reappear with about the same volumetric rate of movement per unit length of shoreline on a beach farther downcoast, where the shore direction again becomes approximately parallel to the wave crests.

Nature provides many instances, however, where drifting sand does not follow the original coastline but strikes out on a new path to form sand spits of various configurations. This usually occurs where the break in the shore alignment is exceptionally pronounced. Such formations may provide clues as to what will happen at the end of a breakwater. Natural spits have been variously classified as cuspate forelands, recurved spits, bay-mouth bars, winged headlands, and so on. D. W. Johnson in his treatise on coastal geology (Shore Processes and Shoreline Development, 1919) depicts many of these formations, and through careful geologic analysis, traces out the history of their development. One of the most striking examples is that of Sandy Hook on the New Jersey coast. This is a compound recurved spit that has been developed over geologic time extending northward into Lower Bay at the mouth of the Hudson River. Wave characteristics in this area are difficult to analyze because of complex refraction effects over the wide continental shelf of the Atlantic coast. The resultant mean direction of wave energy, however, is probably more nearly perpendicular to this spit than parallel to it.

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[a] March, 1960, by Omar J. Lillevang.

[2] Div. Engr., Div. of Small Craft Harbors, Sacramento, Calif.

The recurved end of Sandy Hook, on the other hand, does tend to follow the resultant mean direction of wave energy, although the tip probably hooks back well beyond this alignment. This and other examples in nature indicate that a spit will not always extend continuously in a given direction but may curve back toward shallow water if the original bottom becomes deeper in the initial direction of spit extension. It is probable that the rate of supply of littoral material to an open-end spit may have a considerable effect on the alignment that it takes. A large rate of drift seems to cause the spit to extend, roughly, perpendicular to the mean direction of wave energy for a considerable distance into deep water before it begins to curve back toward shore.

When littoral material first begins to pass around the end of a breakwater, the offshore contours are usually in about the same position as they were prior to breakwater construction. As the shoal begins to form, however, the depth contours seaward of both the breakwater and the shoal tend to move seaward. The rate of movement of these contours is probably considerably slower than the rate of extension of the shoal, especially as it approaches shallow water. A high rate of supply, on the other hand, could cause a more rapid build-up of the offshore contours with a resultant change in the refraction characteristics that affect the approaching waves. It would tend to force the shoal alignment seaward and thus make it more nearly parallel to the wave crests. The great variety of shapes that occur in natural sand spits tends to confirm this supposition. This may be an important factor that has been discounted in the author's analysis of the problem. It is possible that the observed extension of the Santa Barbara shoal in the mean direction of wave energy is merely a coincidence and that the Oceanside shoal will behave differently. At Oceanside, however, the net annual rate of drift is relatively small and the depth at the end of the breakwater relatively shallow, so that extension of the shoal directly toward shore is the more probable result.

It is recognized that most natural sand spits have been formed over a considerable period of time during which the offshore contours have been modified to conform to equilibrium conditions created during a gradual build-up period. Most shoals formed behind artificial structures, on the other hand, have had exceptionally short lives in comparison to those occurring in nature. Moreover, the Santa Barbara spit was removed in its entirety every 2 or 3 yrs prior to the period of build-out depicted in Fig. 5 of the paper. Under these circumstances, the offshore contours did not have a chance to achieve the equilibrium alinement of natural spit build-up. Perhaps this is the secret of coincidence of shoaling direction with mean direction of wave energy, and as long as the tip shoal is removed at frequent intervals, the energy-vector concept may be entirely valid.

The usual solution to littoral by-passing is to intercept the sand at some point before it reaches the harbor entrance and transfer it to the downdrift shore beyond the wave shadow of the harbor structures. The two instances cited, however, may indicate a new trend in littoral drift by-passing procedure, especially in cases where the breakwater is built as an extension of a headland to protect the bay area formed by a reentrant segment of the shoreline. Experience alone will prove the validity of the energy-vector concept as described in this paper. Meanwhile, such factors as bottom depths in the predicted shoal site, settlement rates of the material being transported, rate of transport of littoral material along the particular segment of shore involved, and perhaps other parameters, as well as mean direction of wave energy,

should be analyzed to determine their possible influence on tip-shoal formation. If continued evidence of shoal formation in the mean direction of wave energy proves the author's hypothesis to be correct, even when the above-mentioned parameters vary significantly from those applicable to the Santa Barbara site, the method of analysis presented in this paper will be a welcome addition to man's understanding of shore processes as applied to coastal engineering problems.

PER BRUUN,[3] F. ASCE.—Major contribution to discussion and solution of the littoral drift balance problems based on actual wave data have come from California. Mr. Lillevang's is the latest one and presents one more attempt to secure a useful result. The approach must, for understandable reasons, be of semi-theoretical or "halfway" empirical nature in as much as the basic physical elements are not understood. Earlier California attempts include the "Los Angeles formula" by the U.S. Army Corps of Engineers, Los Angeles, the first robust effort based on actual quantities of material; Munch-Peterson philosophical theory based on wind data only (IV Hydrologische Konferenz, Leningrad, 1933); Bruun's coastal morphological approach based on actual field experiments (1949-1954); and similar theories of mathematical-philosophical nature based on actual observations of free shorelines by Larras (1957) and Thijsse.

The California theories are related to field experience from a number of harbors on the Pacific coast. Their strength is measured by the extent to which these field data are reliable and rightly interpreted.

Mr. Lillevang's says that he "concluded that absence of experimental or observational verification of the Q factor for littoral drift computation (using the Los Angeles formula) did not eliminate it as a useful tool if its empirical parts wholly or nearly disappear by cancellation." The cancellation procedure itself can be discussed and as Mr. Lillevang says in his conclusion, "the mean effective wave energy direction concept and its applications to practical engineering design problems are not pure, in the scientific and mathematical sense." The "resultant wave energy" or "direction of wave energy," the discussions concerning the proper ratio between downcoast and upcoast annual littoral drift quantities, and concerning the adding of the southern swell relative wave energy vector from the adopted 60:40 (ratio) computation to the wave energy diagram could all give rise to some suspicion (or even some severe criticism in as much as wave energy does not have any direction at all). However, Mr. Lillevang would probably ask what would be supplied in its stead, or what is better? The answer must honestly be, not yet. However, we should have something better and probably now is the time when we should follow the advice given in Mr. Lillevang's conclusion: "Try to look deeper into the problem from a physics point of view."

It has already been said that energy (or work) has no "direction." Yet littoral drift must have something to do with energy or to its related longshore "flux." The relation, however, is not intimate. Going deeper into the physical problem, we must consider the important elements of the forces exerted on the bottom by the oscillating wave motion and the longshore current. The main task of the forces is to tear loose the particles from the bottom after

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[3] Head, Coastal Engrg. Lab., Univ. of Florida, Gainesville, Fla.

which the currents transport material away from the scene. The physical approach should probably then be to relate these wave induced forces to the material transport. Many attempts have been made to describe material transport in rivers by shear stresses between the flow and the bottom (Einstein's approach of earlier date, Kalinske's, Meyer-Peter's and Shield's).

We now know that the problem is not simple, but, on the other hand, the shear stress is involved (or the problem can at least to some extent be described by shear stress) in a more complex way. Perhaps we could start on seashores as we started in rivers by considering the shear stress as a pertinent factor. It is certain indication that shear stress has something to do with the actual configuration and stability of the bottom profile.[4] It is natural to take the next step relating the stability of the bottom to the stirring-up (shaving away) of material. First, cut loose the longshore currents (Putnam, Munk and Trayler, 1949) give the "material flux" $\sim \tau$ v, in which v is the longshore transport velocity and the τ v is integrated from bottom to surface replacing τ with a function $f(\tau)$.

Such attempt has not yet been made because it will require a large equipment set-up. It seems to the writer that the first step must be a further clarification of the configuration of the bottom profile in relation to the forces exerted upon it. These forces will vary with depth, wave, and material characteristics. At one depth, bottom may be stable with $\tau = 0.2$ kg/m^2 and at another depth, it is stable with $\tau = 0.5$ kg per sq m (as at tidal inlets in alluvial material). The stability shear stress depends upon the littoral drift transport itself, and its bed load as well as its suspended load and actual stability is a result of a continuous replacement of material in the profile where each section of same respects a certain average "stability shear stress." Perhaps we should go ahead with the adoption of a "shear-stress-times-longshore-velocity" approach in the preliminary stage of further attempts at development.

The above remarks were made upon the suggestion of Mr. Lillevang whose concentrated efforts deserve much respect because of his attempts to find an answer to a problem we face today, and today's problems are important.

J. M. JORDAAN JR.[5]—In his conclusions, the author has invited evaluation and extension of his theory. The writer feels, therefore, tempted to present a somewhat simplified one-dimensional approach that agrees in principle with the author's concepts. A true picture of the three dimensional distribution of sand transport may be obtainable only with extensive model tests, but the following approach may help to reduce the variables.

The writer may also point out that wave direction is readily photographed when the sun is at a low angle and any desired amount of records on wave angle can be easily collected by this means, either by using shore based cameras or aerial photography.

The writer feels that the wave angle at breaking is the governing factor with regard to the sand transported by the uprush but that the longshore current generated by the angularity of approach in deep water governs the transport of suspended sand in the breaker zone. The relative magnitude of the two zones of transport would be amenable to model analysis. The writer feels that W $\sin^2$ I would be more correct than W sin I cos I of the paper.

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[4] Beach Erosion Bd., TM 44.

[5] Head, Hydr. Sect., Natl. Mech. Engrg. Research Inst., S. A. Council for Scientific and Industrial Research, Pretoria, South Africa.

Evaluation of the strength of the longshore current may be approached from two points of view, depending on whether the current is flowing like a river along a straight unobstructed shoreline (true for definite angularities of wave approach) or is building up from null points following rip currents. The latter is the likely condition where the angularity of approach is negligible and the coastline is irregular.

Approach 1: Uniform Longshore Current.—A tractive force F sin α per unit length alongshore acts on the longshore current which is confined largely within and immediately outside the breaker zone. Treating the longshore current as a river with uniform slope sin i, then

$$\sin i = \frac{F \sin \alpha}{\rho\, g\, h\, b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Since the total wave power per unit length is given by $\frac{1}{8}\,\rho\, g\, H^2\, C$, which is equal to a force times a velocity (the wave celerity), the total force due to the waves is

$$F = \frac{1}{8}\,\rho\, g\, H^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

acting in the direction of wave approach. The component of the force alongshore is Eq. 8 times sin α, hence by substitution

$$\sin i = \frac{1}{8}\,\frac{H^2 \sin \alpha}{h\, b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

The mean velocity of the semi-confined longshore "river" may then be obtained by usual formulae, such as Mannings', taking perhaps the hydraulic radius as $\frac{h}{2}$ where h is the depth where shallowing effects begin to occur i.e. $h = \frac{\lambda}{2} = \frac{g\, T^2}{4\, \pi}$, where T is the wave period. The breadth would be the corresponding width. It may be more realistic to take the hydraulic radius as equal to the depth at breaking, equal to 1.5 times the deep water wave height.

Approach 2: Developing Longshore Current.—The energy per foot width of the waves is $\frac{1}{8}\,\rho\, g\, H^2$ per wave, the power dissipated on the shore is then $\frac{1}{8}\,\rho\, g\, H^2\, C \cos^2 \alpha$ and the power generating the longshore current is $\frac{1}{8}\,\rho\, g\, H^2\, C \sin^2 \alpha$, the two (scalar) components totalling to the total power:

$$\frac{1}{8}\,\rho\, g\, H^2\, C,$$

in which ρ is the mass density of sea water, g refers to the gravitational acceleration, H denotes the wave height, trough to crest, C is the wave celerity, and α denotes the angle of approach of waves made with the normal to the coastline.

The non-dissipative portion of the power, acting tangentially on nearshore water over a longshore length L, creates a current with cross-sectional area A and velocity V, given by momentum flux x velocity =

$$\rho\, V^3\, A = \left(\frac{1}{8}\,\rho\, g\, H^2\, C \sin^2 \alpha\right) L \quad \ldots\ldots\ldots\ldots (10)$$

hence

$$V_L = \left(\frac{1}{8}\, g\, \frac{H^2}{A} \sin^2 \alpha\, C\, L\right)^{\frac{1}{3}} \quad \ldots\ldots\ldots\ldots (11)$$

but A is also varying. After the current has grown in width, to the width of the surf zone, the further increase of the longshore velocity with longshore distance is slight and Approach 1 becomes more applicable.

These considerations apply to the mean longshore current water velocity. The littoral sand transport occurs mostly due to wave uprush and due to sand suspension in the current in the breaker zone. With regard to the latter portion, the amount of sand transported due to turbulence is proportional to the current strength and the concentration of sand in the breakers, which is related to the ratio of particle fall velocity to "shear velocity." The latter is again a function of the wave kinematic eddy viscosity, roughly expressible as $\frac{H^2}{T}$. The exact relationships can be determined best by two-dimensional model studies. The distribution of sand transport with distance from shore could be determined by compartmented samplers in a idealized three-dimensional model. It would appear from cursory observations that the material distribution follows the water mean velocity longshore distribution, in other words a "semi Gaussian" distribution with a maximum at the wave uprush, and becoming zero just outside the breaker zone.

It is hoped that these observations contribute something to the author's views and that definitive model studies on littoral transport will be taken up by others also interested in the subject.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158(IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176(WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184(WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205(PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214(SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223(ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240(PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278(HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287(PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296(SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322(ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330(CP2).

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411(SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW1), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436(ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445(EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454(EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478(HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487(CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502(EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511(EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520(PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529(HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(EM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580(EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

SEPTEMBER: 2588(IR3), 2589(IR3), 2590(WW3), 2591(IR3), 2592(HW3), 2593(IR3), 2594(IR3), 2595(IR3), 2596(HW3), 2597(WW3), 2598(IR3), 2599(WW3), 2600(WW3), 2601(WW3), 2602(WW3), 2603(WW3), 2604(HW3), 2605(SA5), 2606(WW3), 2607(SA5), 2608(ST9), 2609(SA5)[c], 2610(IR3), 2611(WW3)[c], 2612(ST9)[c], 2613(IR3)[c], 2614(HW3)[c].

c. Discussion of several papers, grouped by divisions.

PART 2

SEPTEMBER 1960 — 31 NO. WW 3
VOLUME 86 PART 2

Your attention is invited

NEWS OF THE WATERWAYS AND HARBORS DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

JOURNAL OF THE WATERWAYS AND HARBORS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

September, 1960

CHANGES IN COMMITTEE MEMBERSHIP

Dr. Maurice L. Albertson, Director of Research Foundation, Colorado State University, has been appointed to the Division Committee on Regulation and Stabilization of Rivers by Open-Channel Work. Dr. Albertson will fill the vacancy on the Committee resulting from the retirement of Dr. Lorenz G. Straub, Director of St. Anthony Falls Hydraulic Laboratory.

UNIVERSITY OF CALIFORNIA WATER RESOURCES ARCHIVES

As a part of a state-wide University of California research program on water resources, the Water Resources Center Archives on the Berkeley Campus is gathering materials and data in the water resources field. The collection will include all aspects of water: water as a natural resource and its utilization; irrigation; flood control; municipal and industrial water uses and problems; water rights; and water development projects. The emphasis is on material relating to the State of California.

Besides actually acquiring material, the Archives is interested in locating and making biographical listings of important collections which remain in public or private agencies and in engineering offices. A comprehensive catalog will thus be developed for the use of researchers, describing and giving the location of existing unique hydrologic data in the State.

Because the materials found to be the most valuable are not available in the sense that they can be purchased such as most libraries purchase books, the Archives relies upon the cooperation of interested individuals and agencies for augmenting its collection. Engineers' reports and studies (unpublished materials, manuscripts, etc.), agency reports, pamphlets, hearings, legislation, specifications, maps, scrapbooks, newspaper clippings, speeches, campaign material, and books pertaining to water: all are being sought. Interests include both current materials and the historical aspects.

The entire collections of several men prominent in the water resources engineering fields are now housed in the Archives: among these important

Note.—No. 1960-31 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. WW 3, September, 1960

gift collections are those of Frank Adams, Bernard A. Etcheverry, Charles Gilman Hyde and Baldwin M. Woods.

Seven reports have been prepared and published by the Archives, all of them available to the interested public:

1. The Etcheverry Collection in Water Resources Archives (June, 1958).
2. Theses on Engineering, Economic, Social and Legal Aspects of Water (Oct., 1958).
3. Watershed Management Research Data, U. S. Department of Agriculture, Forest Service, California Forest and Range Experiment Station, Berkeley and Glendora, California (Feb., 1959).
4. Publications and Reports of Charles Gilman Hyde (July, 1959).
5. Water Pollution Data, Regional Water Pollution Control Boards, State of California (July, 1959).
6. Bachelor of Science Theses on Water Resources Engineering, University of California, Berkeley (Aug., 1959).
7. Theses on Water Resources, Stanford University, California Institute of Technology, and University of Southern California (Aug., 1959).

The Archives serves researchers in the University community as well as the interested public. The activity is under the direction of Professor J. W. Johnson, Director of Hydraulic Laboratories, University of California, Berkeley. Gerald J. Giefer is the Librarian directly in charge of the Archives.

The mailing address of the Archives is: Water Resources Center Archives, Room 5, Mechanics Building, University of California, Berkeley 4, California.

NEW TASK COMMITTEES

The Division Committee on Regulation and Stabilization of Rivers by Open-Channel Work has formed two new task committees covering the subjects of channel stabilization works and maintenance of navigable waterways.

The Task Committee on Channel Stabilization Works has completed its membership, which consists of the following:

Harvill E. Weller (Chairman)
Supervising Civil Engineer
Lower Mississippi Valley Division
U. S. Army, Corps of Engineers
Vicksburg, Mississippi

Albert F. Bettle, Project Superintendent
International Boundary and Water Commission
United States and Mexico
El Paso, Texas

Francis F. Escoffier, Hydraulic Engineer
U. S. Army Engineer District, Mobile, Alabama

Oliver A. Johnson, Supervising Hydraulic Engineer
U. S. Army Engineer Division, North Pacific
Portland, Oregon

Daryl B. Simons, Research Engineer
Civil Engineering Department
Colorado State University
Fort Collins, Colorado

The purpose of the Task Committee on Channel Stabilization Works is to collect, compile, and analyze data on current practices in the United States, in stabilization of the channels of alluvial rivers, and to report thereon in a series of papers, finally in a summary paper, each to be presented in Convention and published in ASCE Journal with the objective of advancing the knowledge and practice of the profession in this special field. The Committee plans to hold its first committee meeting during the latter part of September, and proposes to present its first papers on the subject at the Phoenix Convention in April 1961.

Mr. Austin B. Smith, Chief, Navigation, Mapping, and Dredging Branch, Lower Mississippi Valley Division, Corps of Engineers, U. S. Army, has accepted the assignment of Chairman of the Task Committee on Maintenance of Navigable Waterways. The Task Committee's purpose is to investigate and report on causes of shoaling in navigable waterways, including shoaling by industrial wastes, the adequacy of dredging methods and equipment, and the design and construction and operation of regulatory structures to prevent shoaling.

STRONG PROGRAM AT BOSTON

Exceptional sessions have been arranged by the Waterways and Harbors Division for presentation as part of the Boston Convention of the Society. This program deserves attention and attendance. While the full program for the convention is available for reference in the August issue of CIVIL ENGINEERING, the division's sessions are shown here:

A Symposium on Hurricane Protection
Presented Monday October 10

- Studies of a Hurricane Barrier in The East Passage of Narragansett Bay - John B. McAleer
- Design of Providence, R. I., Hurricane Protection Project - W. Martin
- Narragansett Bay Studies of Salinity, Temperature and Flushing, and Effects of Proposed Hurricane Barriers - J. Vanderhoeff and Harry Simmons
- Hurricane Parameters in New England Area - Charles Gilman

A Session on Coastal Engineering
Presented Monday, October 10

- Predictions of Slopes of Equilibrium Beaches - P. S. Eagleson
- State Regulation of Coastal Structures - Herbert C. Gee
- Behavior of New England Beach Fills - Harry S. Ferdikie

A Symposium on Materials for Wharf Construction
Presented Tuesday, October 11

- Corrosion of Steel Piles in Salt Water - Mr. Ayres
- Deterioration of Concrete Used for Wharf Construction -
- Damage to Timber Used for Wharf Construction - Shu-T'ien Li

A Session on Regulation and Stabilization of Rivers by Open Channel Work
Presented Tuesday, October 11

- Efficiency of Cape Cod Canal Bank Revetment - H G. Gamble
- Shoaling of Hudson River - Representative from New York Dist., U. S. Army Corps of Engineers
- Latest Dredging Practice - Ole P. Erickson

Control of Ice - Rene Dupuis
Alternate: Chesapeake and Delaware Canal - C. F. Wicker

A Tour of Boston Harbor Facilities
Conducted both Wednesday, October 12, and Thursday, October 13, by boat

FOR YOUR CALENDAR

October 9-13, 1960	ASCE, Boston Convention
April 10-15, 1961	ASCE, Phoenix Convention
October 16-20, 1961	ASCE, New York Convention
February, 1962	ASCE, Houston Convention
May, 1962	ASCE, Omaha Convention
October 15-19, 1962	ASCE, Detroit Convention

NEW DIRECTORY IS AVAILABLE TO MEMBERS

The 1960 Directory is now available to members on request. The Directory lists the entire membership of the Society, giving the membership grade, position, and mailing address of each. In addition, there is a complete listing of the Honorary Members, past and present, and the Life Members. A useful geographical listing of the members is also included.

It goes without saying that the information contained in the Directory is of value to every member, and every member can obtain this valuable information. To receive your free copy of the Directory simply fill out the coupon below. Prompt delivery depends on prompt return of the coupon.

The Society publishes the membership Directory every other year. The next edition will be issued in 1962.

DIRECTORY 1960

ASCE members are entitled to receive, free of charge, the 1960 ASCE Directory. To obtain the directory simply clip this coupon and mail to: American Society of Civil Engineers, 33 West 39th Street, New York 18, N. Y.

Please make the mailing label legible—correct delivery depends on you.

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CHANGE IN PUBLICATION SCHEDULE FOR DIVISION JOURNAL

The quarterly publication schedule for the Journal of the Waterways and Harbors Division is to be changed to the months of February, May, August, and November. In the transition period from the present schedule, the Journal will be issued in September 1960 (this issue), November 1960, February 1961 and so forth.

Material for Newsletter publication should be sent to:

Austin E. Brant, Jr.
Editor, Waterways and Harbors Division Newsletter
Tippetts-Abbett-McCarthy-Stratton
375 Park Avenue
New York 22, N. Y.

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